TM 5-235

DEPARTMENT OF THE ARMY TECHNICAL MANUAL

SPECIAL SURVEYS

HEADQUARTERS, DEPARTMENT OF THE ARMY

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HEADQUARTERS
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SPECIAL SURVEYS

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CHAPTER 1

INTRODUCTION

1. Purpose and Scope

This manual is a reference for engineer officers and technicians responsible for the supervision and conduct of special types of surveys, and describes the instruments and equipment to perform them. Included are—underground, geology and pedology, public and private land, hydrographic, shore-ship, magnetic surveys, and Arctic surveys.

a. Applicability. Information contained in this manual is applicable without modification to both nuclear and nonnuclear warfare.

b. Changes. Users of this publication are encouraged to submit recommended changes or comments to improve the publication. Comments should be keyed to the specific page, paragraph, and line of the text in which the change is recommended. Reasons should be provided for each comment to insure understanding and complete evaluation. Comments should be forwarded direct to the Commandant, U.S. Army Engineer School, Fort Belvoir, Va. 22060

c. Metric Equivalents. The surveys associated with this manual may require using either the metric system or the English system of measurement depending upon the source material and equipment available to the military surveyor. Many of the tables and sample forms given in this manual were prepared under the English system. Where feasible, both English and metric equivalents are given in the text. Conversion factors are listed in appendix II.

2. Special Surveys

The special surveys discussed in this manual are listed below together with their military applications.

a. The Underground Survey. The underground survey pertains to the exploration of caves; the construction of underground shelters; the utilization of underground passages, both man-made and natural; the study of subterranean features; and mining operations under military control.

b. The Geological and Pedological Survey. The geological and pedological survey pertains to the surveyor's functions and responsibilities in support of the geologist, geologic teams, and the soils engineer in the collection of data concerning military engineering projects.

c. Land Surveys. Land surveys pertain to measuring ties to property boundaries and corners; to retracing property lines in connection with a particular mapping activity; and to marking limits of properties under military control.

d. The Hydrographic Survey. The hydrographic survey has the following military applications.

(1) The determination of tidal ranges and the times of diurnal tidal fluctuations. This is essential to construction operations in harbors or tidal waters. The tidal range controls the above-water height of cofferdams and caissons and the cutoff elevations for timber piles. It influences the type of dock construction. The time element in the rise and fall of the tides controls the scheduling of construction operations, as do the range and the times of high and low water. Tidal observations are necessary to determine level datums for bench-mark leveling and to refer soundings taken at different stages of the tide to a common water level.

(2) Measurements of stream flow. These are required in making engineering studies for the development of surface water supplies; in determining adequate waterway openings for bridge and culvert construction; and in obtaining data on stream velocities for river-crossing operations involving pontons, ferries, or permanent bridges.

(3) Soundings required to determine water depths offshore and in rivers and harbors for construction and other engineer activities.

(4) Various hydrographic measurements. These are required for determining the quantities of dredged material; for computing reservoir capacities; and for procuring data required by hydrology teams.

e. Shore-Ship Survey. Field applications have indicated that by interweaving a series of simultaneous observations between points on shore and a signal mounted on a ship offshore, positions can be carried along a coastline rapidly and economically. This method is referred to as the shore-ship method of triangulation.

f. Magnetic Survey. Magnetic surveys are conducted to measure the strength and direction of the earth's magnetic field at specific points on or near the surface of the earth. The earth's magnetic field has an irregular distribution and varies with the passage of time. Magnetic measurements must therefore be widely distributed and repeated periodically. Magnetic data are required for the preparation of improved navigation and world magnetic charts, and for other commercial, scientific and military uses.

g. The Arctic Survey. The Arctic survey pertains to the use of the winterization kit and problems in surveying in the Arctic.

3. Responsibilities

It is the responsibility of the commander of a survey unit to furnish the necessary supervision and operational control. Personnel experienced in the particular type of survey must be made available to conduct the survey as a special task assignment.

a. Military surveys are conducted under two general situations—

(1) In support of tactical operations and field exercises, in which case orders and logistic support follow the normal procedures of *b* below.

(2) In conjunction with nontactical activities involving engineering projects, in which case the survey is frequently conducted as an exempted activity under the Chief of Engineers and is handled as in *c* below.

b. In tactical situations, surveys are directed and administered by tactical commands. Survey activities are directed by operational orders; administrative matters are processed according to administrative memoranda. The operational supervision and logistic support of a survey unit are similar to the command and supply of a combat unit. For field service regulations and staff procedures, see FM's 100–5, 100–10, 101–5, and 101–10.

c. In nontactical situations, special surveys are frequently implemented by means of fund-supported projects. Before funds can be expended, a project must be authorized by a major command, be given an official project title and identification, and be assigned an appropriation symbol. Included in the instrument of approval is the project description which provides information on objectives and requirements; appended are bills of materials, citations of funds, and authorities for personnel.

d. In a nontactical situation it is the responsibility of the survey commander to organize the work according to the basic elements of the project description and to manage survey activities within the limitations of authorized funds. It is a survey function to transcribe the project description into operational orders and appended matter into technical instructions and administrative directives. Included in the administrative directives are instructions for expending funds, for obtaining personnel, and for procuring supplies and services.

e. The following serves as a guide or check-list for supervising a fund-supported project in a nontactical situation.

(1) Manage appropriated funds and nonappropriated funds according to current regulations; use prescribed forms for transactions and records.

(2) Request the appointment of a purchasing and contracting officer to handle procurement; an agent finance officer to disburse funds; and a transportation officer to issue bills of lading and travel requests.

(3) Expend funds of the various technical services for the items for which they are provided. Examples are—quartermaster funds for rations, gasoline, and oil; ordnance funds for emergency repairs to ordnance vehicles; transportation funds for travel of personnel and transport of equipment; and engineer funds for emergency items of engineer supply. The use of credit cards for gasoline and such other authorized purchases lessens the need to handle currency.

(4) Arrange for maintenance of equipment. A regular schedule should be set up for this purpose.

(5) Take periodic inventories of supplies and equipment.

(6) Comply with civilian personnel regulations in the employment of United States civilian personnel and with the labor regulations of foreign countries when employing native personnel abroad.

(7) Use per diem fund, if authorized, to minimize the use of equipment connected with billeting, rationing, and transporting personnel or material when individuals cannot be adequately supported by the central installation of the unit.

(8) Request appropriate authorization before requisitioning items of supply not listed in the bills of materials.

(9) Provide for medical services (immunization, physical examinations, health clearances).

(10) Arrange for the training of equipment operators; for the issuance of drivers' licenses; and for the procurement of such vehicle license tags as are required by law.

(11) Arrange for morale-building activities, including prompt mail service, motion pictures, athletic equipment, library service, barbers, and recreational visits. The more isolated the areas of survey, the more important these items become.

(12) Anticipate the need for guides, pack animals, powerboats, and local labor.

(13) Establish standing operating procedures for routine matters such as making and breaking camp, receiving official visitors, releasing information to the public, and procuring permission from property owners to enter private property.

(14) Establish emergency instructions for probable contingencies or eventualities (accidents, illnesses, deaths).

(15) Arrange for passes into restricted areas and schedule survey operations with regard to range firing or other influencing factors.

(16) Consult the administrative paragraphs of this manual for instructions peculiar to each particular type of specialized survey.

CHAPTER 2

UNDERGROUND SURVEYING

Section I. OPERATIONS

4. Introduction

a. Characteristics. Underground surveys are essentially similar to three-dimensional surveys on the surface in that the purpose of all angle and distance measurements is to obtain the horizontal and vertical coordinates of a point, the positon of which is unknown, with respect to a point of established location.

b. Peculiarities. The following peculiarities of underground surveys indicate how they differ from surface surveys.

(1) The lighting in underground passageways is generally poor and artificial illumination must be provided to view instrument crosshairs, to read verniers, to sight targets, and to permit normal movements of survey personnel in executing their duties.

(2) The working space in passageways is often cramped.

(3) In certain types of operations, survey lines must be carried through locks into pressure chambers.

(4) In many instances the underground workings are wet, with considerable water dripping from the roofs of passageways and running along the floors.

(5) Instrument stations and bench marks for leveling must often be set into the roof of a passageway to minimize disturbance from the operations being carried on in the workings.

(6) Instrument stations are set with some difficulty since plugs must be driven into drill holes in rock.

(7) Lines of sight are frequently very short, either because of crooked passageways or because alinement must often be brought down from the surface through small shafts. Care must therefore be taken in all surveying operations involving the alinement of tunnels or the running of underground traverses.

(8) The sights taken in shafts and sloping passageways are often sharply inclined. It is frequently necessary to observe both horizontal and vertical angles through a prismatic telescope or eyepiece or through an auxiliary telescope mounted either above the main telescope or to one side of the instrument standards.

(9) It is much more difficult to keep satisfactory survey notes when the workings are wet or dirty and the illumination is poor.

(10) Personnel and equipment must be protected from the hazards inherent in many subsurface operations.

(11) The survey must often be performed in a restricted space at the same time that other activities are in progress.

(12) Great care must be taken to keep surveying instruments in adjustment and in proper working order under the adverse conditions so frequently encountered underground.

(13) Precision measurements are more difficult to accomplish than in surface surveying; therefore the surveyor must be particularly alert to provide check measurements wherever possible.

(14) Plumbing down a shaft constitutes a

special problem which is peculiar to underground surveying.

(15) Two vertical dimensions, from a line of sight to both the floor and ceiling of the passage, are involved in underground surveying whereas only one vertical dimension is normally encountered in surface surveys.

5. Application

Subsurface surveys are required for—

a. The construction of tunnels and other underground structures and utilities.

b. The determination of the extent and characteristics of natural caves which may be developed as shelters for military personnel, storage, and operations.

c. The location of existing underground utilities under certain conditions.

d. The rehabilitation of tunnels, underground structures, and utilities damaged by enemy action.

e. The determination of the extent of groundwater horizons.

6. Technical Instruction

Personnel engaged in underground surveys should be well versed in the basic procedures of surface surveying (TM 5–232). They should receive additional instruction on the instruments, methods, and safety precautions peculiar to underground surveys.

7. Size of Party

The majority of survey operations underground are essentially similar to comparable surface surveys. The minimum size of party and the duties of party personnel are not, in general, different from surveys of the same type above ground (TM 5–232). Under certain conditions the underground situation may require additional personnel. For example, differential levels are often run on the surface by an instrumentman, keeping his own notes, and a rodman. In leveling underground, where bench marks must be set on plugs driven into drill holes in the roof of the passageway, one or more drillers, depending upon the hardness of the rock, will greatly expedite progress. Notes can also be kept cleaner and drier if they are the responsibility of a recorder who has no other duty. In a tunnel under construction, two flagmen or signalmen may be required to provide proper protection to the survey party. Additional personnel may be necessary to install roof supports, ventilation equipment, and the like in caves or abandoned underground workings. Certain rough surveys can be made by one man but, where exploratory surveys are made in caves or abandoned underground passageways, it is prudent to employ two or more men for greater safety. Personnel must be not only well trained, but agile and keenly alert.

8. Coordination With Construction Operations

General command responsibilities applicable to survey operations of all types are itemized in paragraph 3. The chief of the survey party is responsible for the proper coordination of the survey work with subsurface construction operations and with matters of safety. Coordination measures will include arrangements with the mine or construction superintendent to stop operating lifts during a specified period for the purpose of plumbing down shafts, and to schedule survey operations for such hours or in such parts of the workings as will result in minimum interference between the survey work and other concurrent underground operations.

9. Safety and Health

Injuries and fatalities in underground survey operations may result from a variety of causes. Suitable precautions must be taken to minimize the hazards and to prevent accidents.

a. Accidents. The major causes of accidents include—

(1) Falling down shafts or steeply inclined passages.

(2) Tripping over obstacles in inadequately lighted workings.

(3) Being struck by objects dropped from above.

(4) Being struck by falling rock or ore from the roof or wall of a passageway.

(5) Explosions of methane (CH_4), coal dust, or other inflammable gases or substances.

(6) Fires resulting from explosions or from other causes.

(7) Improper operation or inadequate maintenance of subsurface transportation equipment or of mining or construction machinery.

(8) Stepping on nails.

(9) Slipping on muddy slopes.

(10) Striking head in low-ceilinged areas.

(11) Electrocution from contact with exposed wiring.

b. Health Hazards. The major health hazards include the breathing of noxious or poisonous gases such as carbon monoxide (CO) and the breathing of siliceous rock dusts which may result in silicosis. Certain other diseases are commonly associated with underground operations but their incidence may be minimized by good sanitary practices in the workings. Caisson disease must be guarded against in compressed-air tunneling or foundation work.

c. Accident Prevention. Many accidents may be prevented by taking adequate safety precautions. Certain of these precautions are within the capabilities of the survey unit and should be under the direct control of the unit commander. Other precautions apply to underground construction operations and to the use of underground passages for the shelter of personnel or for other military activities. Precautions and safety measures to be taken include—

(1) Issuing protective clothing including helmets, safety shoes, goggles, and dust masks.

(2) Providing wire-mesh fences and gates at the tops of shafts and at all underground levels.

(3) Adequate safety organization, education, discipline, supervision, and inspection.

(4) Using carbon monoxide and methane detectors.

(5) Testing the top and walls of a passage with a spalling bar before working in an area.

(6) Using timbering or roof bolting liberally in areas where the top of a passage is in a zone of weakness.

(7) Keeping surveying instruments, tapes, and other metallic equipment clear of trolley wires or other exposed wiring.

(8) Keeping the working area clear of protruding nails, scrap lumber, broken rock, and other materials.

(9) Cautioning personnel wearing eyeglasses or protective goggles that fogging of lenses may occur quickly upon going underground or when doing strenuous physical work. The wearer must stop at once to wipe the lenses clean to eliminate the dangers resulting from obscured vision.

(10) Using adequate and nonflammable illuminating devices.

(11) Using well-constructed ladders, guardrails, and hand ropes to facilitate safe operations in caves and other underground passages.

(12) Having supplementary light sources in case of failure of primary lights.

d. Additional Health and Safety Measures. In addition to the above precautions the construction and operation of underground facilities will require other health and safety measures which should be checked on by the survey commander where they affect survey operations. These include—

(1) Man cages in shafts provided with strong bonnets, enclosed sides, gates, and safety catches.

(2) Adequate and controlled ventilation of the underground workings to eliminate carbon monoxide, methane, and other poisonous or explosive gases and to reduce the high temperatures and humidities encountered in deep workings.

(3) Fireproof construction in critical locations such as shaft headings; fire doors and stops; and firefighting equipment and personnel.

(4) Wetting and rock dusting to minimize the danger of coal-dust explosions.

(5) Wet-drilling methods where siliceous dusts are encountered.

(6) Adequate drainage of the workings.

(7) Properly controlled storage and use of explosives.

(8) Sanitary change rooms and shower rooms.

(9) Adequate toilet facilities both above and below ground.

(10) Organized rescue crews and well-equipped rescue stations.

(11) A hospital lock on compressed-air operations.

(12) Adequate first-aid and medical facilities.

10. Required Surface Surveys

In connection with the majority of underground surveys, control surveys are required on the surface to determine the position and elevations of points at the tops of shafts, at portals, adits, and boreholes. These points are then transferred underground by instrument, by tape, and by shaft-plumbing. Thus the stations and lines of the underground surveys are tied in with located surface points and are controlled in position, elevation, and azimuth.

a. Establishing Surface Control. Methods of plane surveying (TM 5–232) are suitable for establishing surface control for the majority of underground surveys. Control surveys for long tunnels may require the application of precise triangulation and leveling procedures (TM 5–441).

b. Purpose of Surface Survey. The surface survey is required to provide data necessary to locate, define, or establish the boundaries of the survey area and the underground openings, or to control the alinement of a tunnel. Figure 1 shows typical triangulation control for a projected tunnel *AB*. By extending triangulation from a measured base *AC* to a check base *BD* it is possible to compute the length and azimuth of the proposed tunnel. This azimuth is run out over the ground surface, as indicated by the small flags in the profile view, to locate construction shafts at *E* and *F*. Differential levels run between portals and shaft headworks will fix the grade of the tunnel and the depth of each shaft. The azimuth of the tunnel is transferred down each shaft by plumbing and into each portal so that the tunnel excavation can proceed, as indicated by arrows in the profile view, on the proper line and grade with a heading at each portal and two at each shaft. The portals may be connected by traverse rather than by triangulation. The distance between plumb wires in the shafts is seldom over 5 meters (15 ft) whereas the shafts and portals may be several miles apart. It is clear that all control surveys must be carried out with the utmost care to assure holding through the several headings on the proper line and grade. Many other applications of surface control are required in underground work, but the above example indicates the purpose of surface control and the general method of transferring control data to the underground survey system.

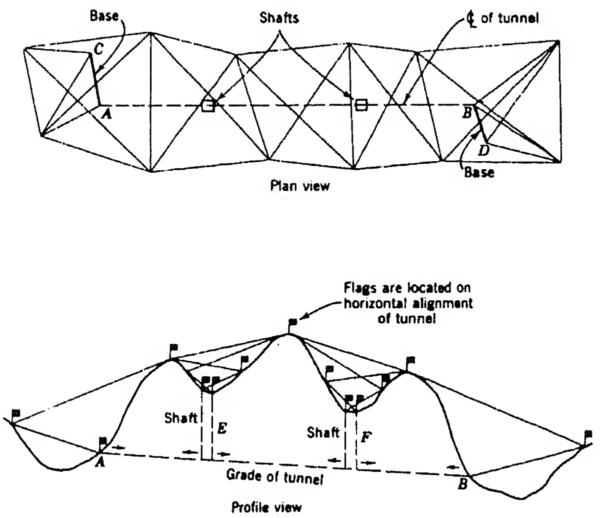


Figure 1. Triangulation control for a projected tunnel

Section II. EQUIPMENT

11. Engineer Items of Issue

a. Many items of military issue are suitable for use in underground surveys. The following is a typical list of engineer-issue items which meet the general requirements of such surveys.

Book, note (level, topographic, transit)
Clinometer, service

Collimator, vertical, Parkhurst type
Compass, forestry
Compass, lensatic
Compass, surveying
Lamp, carbide, hand (for mine use)
Lamp, signal, survey, electric
Lantern, electric, portable
Level, dumpy, engineer
Level, hand, Abney
Plumb, bob, 10-, 16-, 32-oz
Rod, level, Philadelphia
Tape, measuring, band-chain, steel, 30m, 50m 100-, 300-foot
Target, Philadelphia rod, micrometer
Theodolite, direction
Transit, engineer
Transit, engineer, night-illuminated
Tripods, trivets, and base plates for instruments issued.

b. The execution of special surveys may require items of equipment not listed in Department of the Army supply manuals. In this case it will be necessary to obtain authority to purchase such items. For example, when working in close quarters, a special extension-leg tripod (3 feet—approx 1 meter—in length and closing to 2 feet—$\frac{2}{3}$m) may be preferred to a fixed-leg tripod or to the usual extension-leg tripod (5 feet—$1\frac{1}{2}$m—in length and closing to 3 feet—1m). The following is a typical list of such items:

Bracket
Clinometer, hanging
Compass, Brunton
Compass, hanging
Illuminating devices
Micrometer head (or lateral adjuster)
Plumb-bob dampeners
Rod, level, extension (5 ft closing to 3 ft)
Scales, brass
Spads
Telescope, auxiliary (for transit use)
Transit, mining
Tripods, extension (3 ft closing to 2 ft)
Trivets, tunnel

12. Requirements of Underground Surveying Instruments

Underground surveys can be performed with standard transits or levels designed for surface surveys. Instruments used underground should have a centering point on top of the main telescope or on the upper post of the telescope axis to permit setting up under overhead stations. A mining transit or a transit with attachments is necessary for the measurement of both vertical and horizontal angles when the line of sight is inclined steeply either upward or downward. A short-focus attachment is useful for shaft plumbing. A built-in lighting system for the crosshairs and the horizontal and vertical circles facilitates and speeds survey operations.

13. Standard Instruments

Any standard military transit is adaptable to underground surveys except for those angular measurements involving steeply inclined sights. These instruments are not equipped with auxiliary or prismatic telescopes. The direction theodolite (para. 11*a*) with elbow eyepiece attachment can be used for steeply inclined, or vertical, upward sights. The characteristics and features of standard military instruments compare favorably with those listed for mining transits in paragraph 15.

14. Mining Instruments and Attachments

a. *Elbow Eyepiece*. This eyepiece (fig. 2) permits the sighting of positive vertical angles up to 90°. The eyepiece can be attached to standard-issue instruments which would otherwise be limited to the measurement of positive vertical angles of approximately 60°.

b. *Auxiliary Telescope*. The auxiliary telescope, mounted either above the main telescope or on the side of the standards (fig. 3), allows vertical or steeply inclined sights to be taken either upward or downward. The balance of

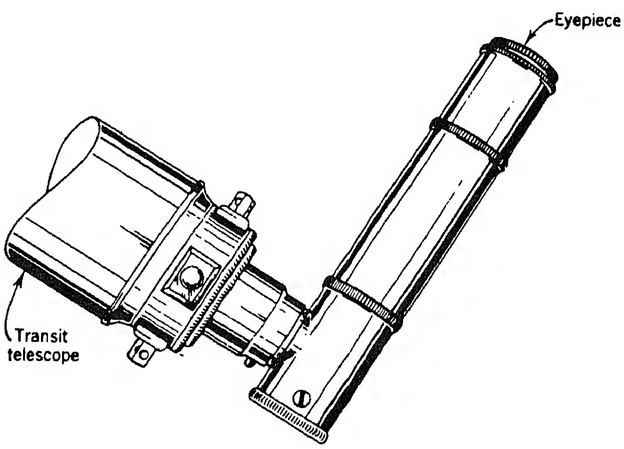


Figure 2. Elbow eyepiece.

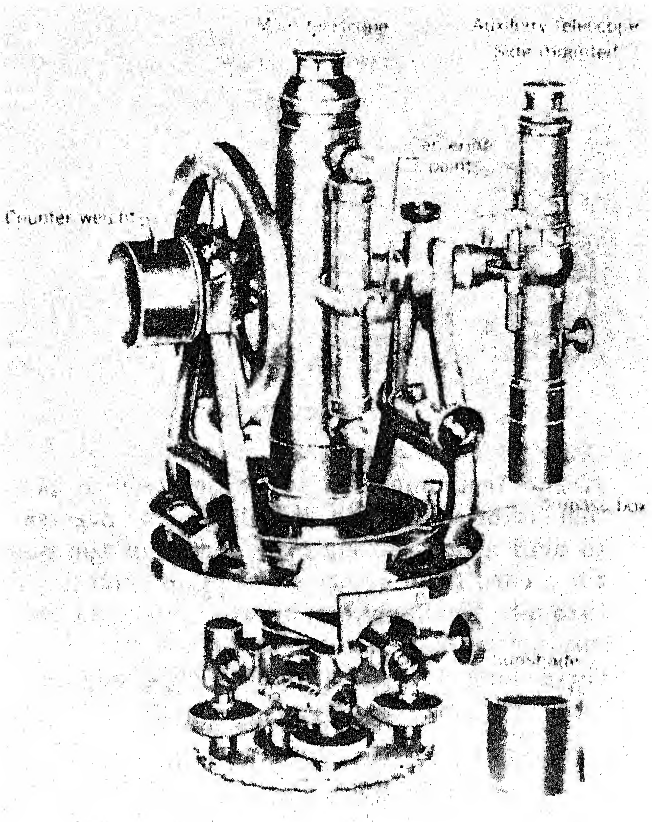


Figure 3. Mining transit with auxiliary side telescope.

the instrument is maintained by attaching a counterweight on the opposite end of the horizontal axis when the auxiliary telescope is mounted in the side position, or by attaching the counterweight under the main telescope when the auxiliary telescope is mounted above the main telescope. The ordinary transit is limited to the measurement of vertical angles of 60° or less; for larger angles the horizontal plate interferes with the line of sight.

c. Prismatic Telescope.

(1) *One prism.* A single 90° prism is attached to the objective end of the telescope, permitting either upward or downward sights throughout the entire vertical range.

(2) *Two prisms.* The two-prism instrument has a hollow horizontal axis with an objective prism attached to the right end (as the instrument is viewed from the eyepiece end of the main telescope). Light from the target sighted deflected by this prism through the hollow horizontal axis to a second prism in the telescope which deflects the light through the main telescope to the eyepiece. This second prism can be shifted laterally by means of a knurled knob to permit direct shifts through the main telescope when large vertical angles are not involved.

d. Short-Focus Attachment. This attachment (fig. 4) is fitted to the objective end of the instrument the same way as the sunshade and dust shield. It makes possible a minimum focus (71 cm) 28 inches from the center of the telescope.

15. List of Characteristics for Mining Transits

Transits used in underground surveys should have the following characteristics in addition to meeting the requirements of paragraph 14.

a. Large diameter, full vertical circle rather than a vertical arc.

b. Two verniers on opposite sides of the vertical circle to eliminate eccentricity (optional).

c. Edge graduations on vertical circle instead of face graduations to permit reading vertical angles without moving from behind the instrument in a poorly-lighted or cramped setup (optional).

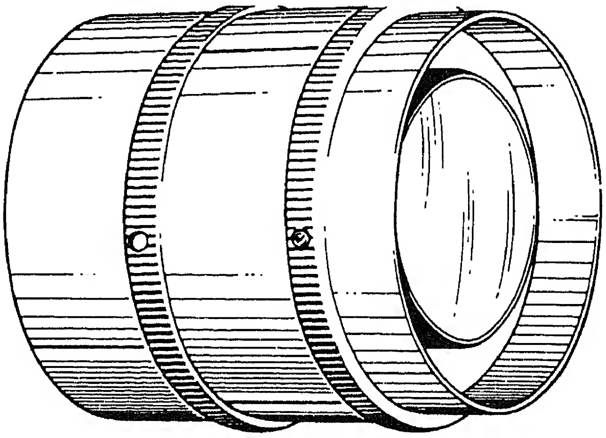


Figure 4. Short-focus attachment.

d. Interchangeable side or top auxiliary telescope or a prismatic telescope for steeply inclined sights.

e. Striding level for leveling horizontal axis when steeply inclined sights are being taken.

f. Interior focusing system (water- and dust-proof qualities).

g. Lighting system for crosshairs and for horizontal and vertical circles (optional).

h. Inverting optical system to provide maximum light for sighting.

16. Transit Mountings

a. Tripods. The standard transit tripod having legs of fixed length should be used where possible since it provides a more stable support for the instrument. In low underground passages, it is often necessary to mount the instrument on an extension-leg tripod. The usual extension-leg tripod has a length of nearly 1.5 meters (5 ft) and closes to about 1 meter (3 ft). In close quarters, a special extension tripod

[illegible] which [illegible] used in surface [illegible] on the parapet wall [illegible] on the top of a triangulation tower. Many situations arise in underground surveys where a very low setup is necessary or where the instrument must be placed upon a heavy cross timber. The military theodolite normally is provided with a trivet. The metal base plate for a military transit can be transformed into an improvised trivet by tapping three threaded holes at appropriate positions in the plate into which bolts can be fastened to provide a three-point support for the plate.

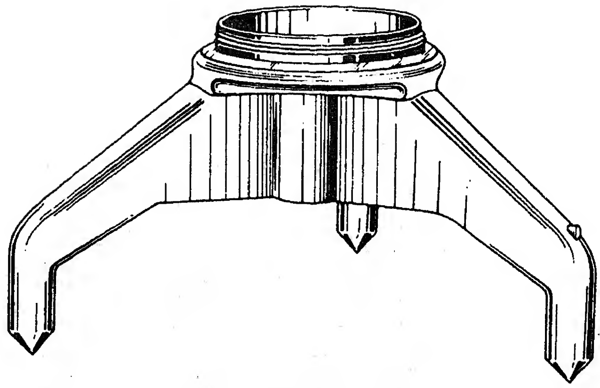


Figure 5. Trivet.

If the instrument is to be set over a point instead of under one, it will also be necessary to drill a hole through the center of the plate for a cord from a very short plumb bob to pass through. Similarly, an improvised trivet for a mining transit can be fashioned by passing three long screws through the wooden base plate to provide support.

c. Brackets. In a timbered underground passage a tripod setup may interfere with construction or other operations. There are also places where a setup very close to a wall is necessary to carry a traverse around a corner in the passageway. For these conditions a bracket (fig. 6) provides a convenient instrument support. A hole is drilled horizontally into a sound timber at the proper location and the steel bracket bar is screwed firmly into this hole. The bar carries a screw plate for the instrument which is leveled by means of the usual leveling screws. Then by means of the sliding head it is brought under the plumb bob hung from the overhead station point. The station point must have been so set as to make this operation possible. When a bracket setup is desired in an untimbered passage, either a wood post can be wedged between roof and floor or a wooden plug can be driven into a drill hole in the wall to provide the desired support.

d. Micrometer Head. The micrometer head or lateral adjuster (fig. 7) is a useful attachment for centering an instrument on line between two points, for alining an instrument with two wires hung in a shaft, or for centering the transit under an overhead station. The attach-

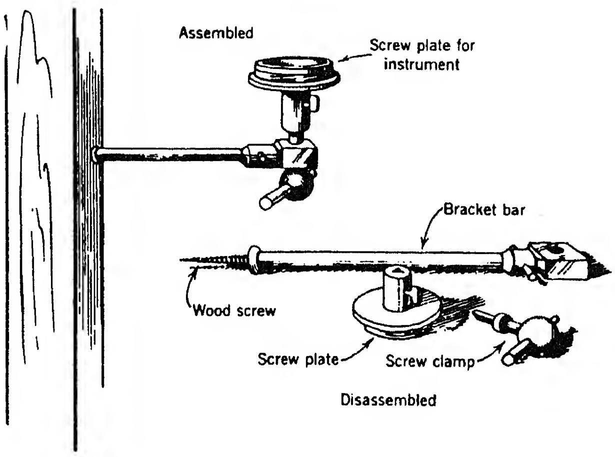


Figure 6. Bracket.

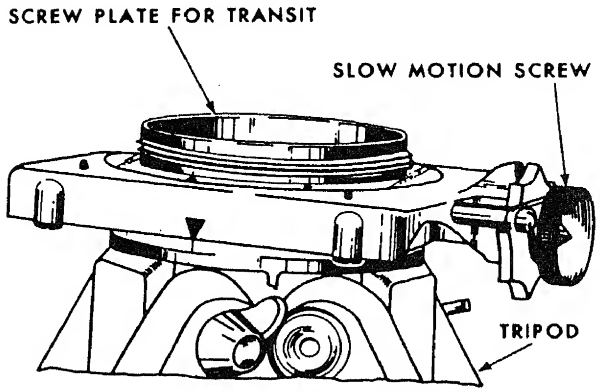


Figure 7. Micrometer head or lateral adjuster.

ment is threaded on its underside to fit onto a tripod, a trivet, or a bracket and it carries threads on top to receive the base of the transit. Lateral movement is regulated by a slow-motion screw. The device permits a greater horizontal movement than the usual shifting head of the transit and the movement can be more accurately controlled.

17. Compasses and Clinometers

Horizontal and vertical angles are frequently measured by compass and clinometer in underground reconnaissance surveys or for rough measurements. Local attraction of the magnetic needle is frequently encountered underground, making magnetic gearings unreliable. If a compass is held or hung directly below a station and bearings are read forward to the next station and back to the previous station of the traverse, the horizontal angles computed from these bearings will be free from the effect of local attraction. The military-issue clinometer and compasses listed in paragraph 11*a* are suitable for such measurements. Other special instruments which may be procured or improvised for use on underground surveys are described below.

a. Hanging Compass. The hanging compass (fig. 8) is extremely useful for rough traverses through crooked passageways. In caves or passages where no local attraction can be detected, a strong cord is stretched from station to station, the compass is hung in gimbals on this cord, and the bearing of the line is read. Where the card is on a considerable slope, an improvised clip or stop is necessary to keep the compass from sliding. In passages where there is local attraction, a special appliance is needed to permit hanging the instrument directly under the station so that both forward and back bearings can be read from the same position of the instrument.

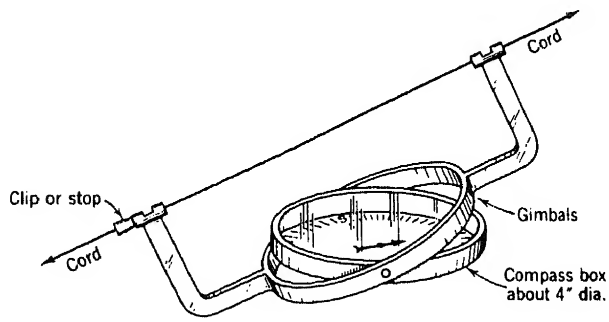


Figure 8. Hanging compass.

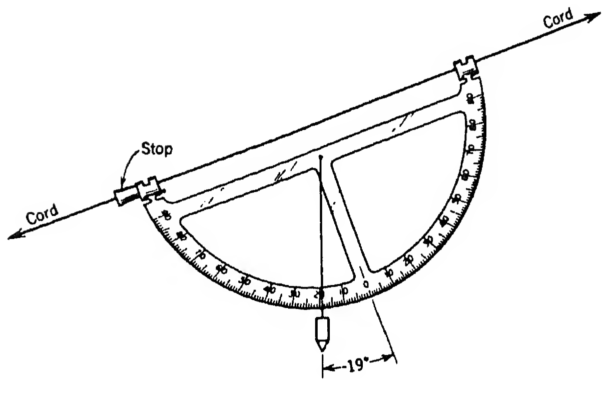


Figure 9. Hanging clinometer.

b. Hanging Clinometer. The hanging clinometer (fig. 9) is a companion instrument to the hanging compass. The compass measures the bearings of traverse lines, or provides data for computing the horizontal angle at a traverse station, and the clinometer measures the angle of slope of a cord stretched between two stations. Because of the weight of the clinometer and because the stretched cord hangs in a catenary, two slope readings are generally required. These are read at points equidistant from each end and as close to the stations as possible. The slope is taken as the mean of the two readings. When the slope of the cord is relatively flat (less than 10°), a single reading at the center of the span will give the angle within the limits of accuracy of the instrument.

c. Brunton Pocket Transit. This instrument (fig. 10) is a combination compass and clinometer. The instrument can be mounted on a light tripod or staff or it may be cradled in the hand. The cover of the compass box is hinged

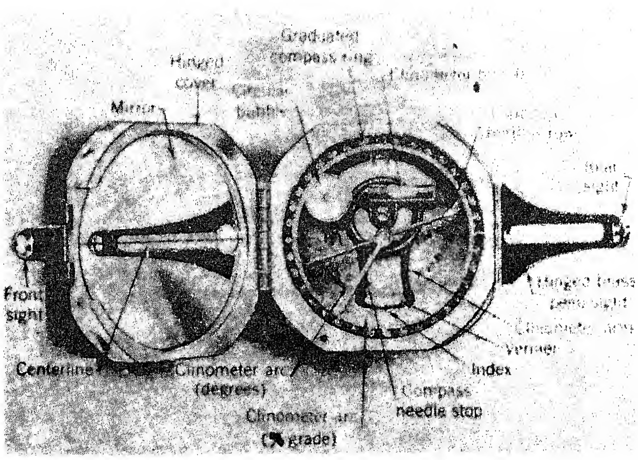


Figure 10. Brunton pocket transit.

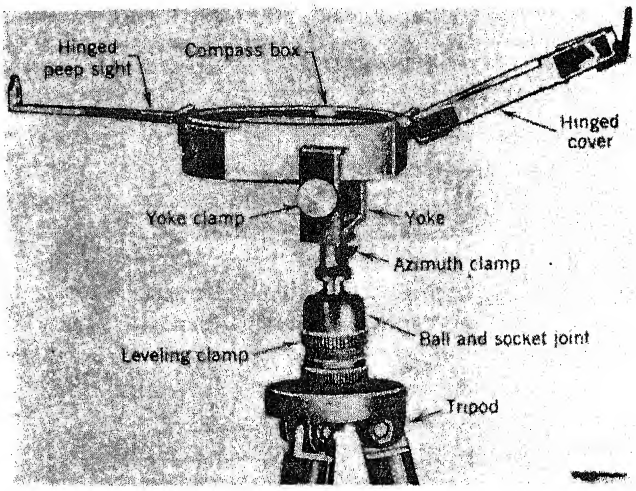


Figure 11. Tripod mounting for Brunton pocket transit.

the instrument is held or mounted with the compass ring horizontal, a circular bubble being used for leveling. The hinged cover and peepsight are raised to such a position that the observer can see both the point sighted and the reflected image of the compass reading. Used as a clinometer, the compass box is placed in a vertical plane with the clinometer bubble on top and the clinometer arcs below. The peepsight holes are used for sighting along the slope to the station ahead and, when the clinometer bubble is level, the reading of the index of the clinometer arm is taken. One clinometer arc is graduated in degrees, and readings to 5 minutes are obtained from a vernier on the clinometer arm. The lower arc is graduated in percent of grade and is read opposite the index of the arm. The instrument weighs about 0.28 kg (one-half pound) and is easily carried in a pocket. When it is to be mounted on a tripod (fig. 11), the pocket transit is clamped in a yoke which is supported, through a ball-and-socket joint, by a head fitting on a small tripod. Knobs are provided on clamps so that the instrument, when leveled, can be clamped in position on the desired line of sight. A forester's compass is similar to the Brunton pocket transit but simpler in design. It consists of a compass ring graduated from 0° to 360° and a simple clinometer.

18. Levels and Level Rods

a. Levels. Levels suitable for differential leveling in underground surveys include: the

military engineer level, ES 42–74; the dumpy engineer level; and the wye engineer level. The Abney or Locke hand level is suitable for rough measurements of differences in elevation. Small extension-leg tripods, brackets, and trivets are useful for setting up a level underground.

b. Level Rods. Level rods for underground work are similar to those used in surface surveys but, because the headroom is usually limited, shorter rods, 1 to 1½ meters (3 to 5 feet) in length, are frequently used. These rods should be designed to receive the standard micrometer target for a Philadelphia rod since, in poorly lighted passages, it is impossible to read the rod from the instrument and sights must be taken on an illuminated target.

19. Tapes

Tape measurements in underground surveys are often made on a slope, the distance being measured from the centering point on the transit telescope to the station sighted and the horizontal distance computed by use of the vertical angle measured. It is therefore important that an underground survey party be provided with a tape of sufficient length to measure the longest slope distance required. The 30m, 50m 100-, and 300-foot steel tapes (para. 11) are suitable. In addition, the 50-foot, woven-metallic, measuring tape is useful for measuring instrument heights, station heights, and short offsets. White-faced tapes, when procurable, are useful because they are easily read in dimly lit passageways and because the coating material used will protect the steel band from the corrosion of acid mine waters. On the other hand, these coated tapes are more readily corroded by the waters in limestone caves than are the etched tapes. The military-issue tapes, if kept properly cleaned and oiled, will give good service in underground surveys. Clamping handles and spring balances are necessary for precision measurements. Horizontal measurements are possible in relatively level drifts by placing a small wooden slide or other target on the cord of the sighted plumb bob and setting the slide at the level of the instrument telescope. The tape is then stretched between the centering point of the telescope and the slide on the plumb bob cord. The tapes used should be kept wound on reels when not in actual use to minimize breakage in the dimly lighted passageways in which other military operations may be taking place. The survey party should be equipped with a tape-repair kit for mending broken tapes and should be amply supplied with cleaning rags and oil so that the tape can be thoroughly cleaned and oiled after use. In wet passages, a rough wiping may be all that is possible underground. The reels should therefore be open and of ample capacity since some moisture and dirt will be wound in with the tape under these conditions. When the party reaches the surface, the tape should be removed from the reel and both tape and reel should be thoroughly cleaned and oiled.

20. Illuminating Devices

a. For Targets. The devices used for illuminating targets in underground surveys include—

(1) White cardboard placed behind the string of a plumb bob hung from the station sighted. This is satisfactory when the sight is short and the passage is well lighted.

(2) Standard-issue signal lamps. These include both the 50- and 127-mm (2- and 5-in.) diameter lamps. A signal lamp can be held in such a position as to illuminate a target, or it may be placed directly on the line of sight so that the telescope of the instrument is focused on the lamp bulb.

b. For Crosshairs. A shade tube provided with a diagonally set annular mirror (fig. 12) can be placed on the transit telescope in place of the regular sunshade tube. A signal lamp is then held to one side of the telescope in such a position that enough light is reflected down the telescope to illuminate the crosshairs. It is also possible to shine light down the regular sunshade tube with a lamp, but more satisfactory illumination is obtained if a piece of tracing cloth or paper is rolled into a tube and inserted in the sunshade. This will reflect enough light from the lamp to illuminate the crosshairs. The standard military direction theodolite has illuminated crosshairs.

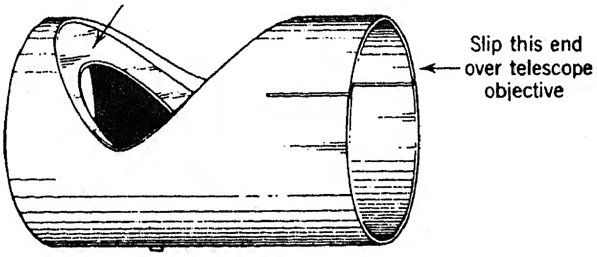


Figure 12. Reflector shade tube.

c. *For Measurements.* Electric signal lamps normally are used for reading tapes and verniers. The direction theodolite and the night-illuminated engineer transit are provided with built-in illuminating devices for reading both the horizontal and vertical circles. Care must be taken to minimize local attraction when a signal lamp is used to read a compass.

21. Accessories

a. *Rods and Targets.* Level rods and targets for underground work are mentioned in paragraph 18. Short range poles are sometime used for giving line but, since most instrument sta-

must be heavy enough to straighten the wire used for the line. They will vary in weight from 4.5 to more than 45 kg (10 to 100 lbs). Brass casings filled with lead, or a section of I-beam or other symmetrically shaped structural section may be suitable. Heavy bobs are also designed so that their weight can be varied by screwing threaded sections on or off. These heavy bobs are often fitted with vanes or fins to reduce twisting and vibration. The bobs are immersed in a container of water or oil to reduce swing and vibration. Crankcase and heavy transmission oils are readily available for such use. The container should be covered, except for a small hole for the wire to pass through, to avoid disturbance from dripping

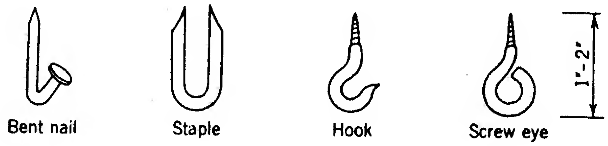


Figure 13. Improvised spads.

water. A large drum partially filled with oil is preferred.

c. *Spads and Plugs.* Permanent stations normally are set in underground passageways by drilling a small hole, 12.5 to 25-mm (½ to 1 in.) in diameter, 38 to 50-mm (1½ to 2 in.) into the rock of the roof. A plug, consisting of a piece of softwood dowel or a roll of soft lead sheet, is driven into the drill hole until it is flush. A spad, to hold the plumbbob cord, is then driven into the plug. Spads can be improvised by using a bent nail, a staple, a hook, or a screw eye (fig. 13). These should penetrate at least 12.5 mm (½ in.) into the plug. Galvanized nails, staples, or screw eyes and brass hooks will last longer in wet passageways than those of plain iron or steel. The open hook is more efficient than the staple and the screw eye, which require the bob cord to be threaded through the eyelet. Where greater permanency is desired, special spads of brass, copper, zinc, chrome-steel, or bronze can be used.

d. *Brass Scales.* In plumbing down shafts, a scale of brass or other suitable material (fig. 14) is commonly fastened to the timbering near the bottom of the shaft immediately behind the wire which supports the heavy plumb bob. As the wire swings or vibrates gently, its extreme positions are read on the scale and the mean position determined. A vernier rider carrying a small plumb bob or target is then set on the scale at this reading and used as a backsight for a transit at the bottom of the shaft. Scales inked on a translucent screen and illuminated by a light placed behind it are also used for this purpose, and for establishing tunnel alinement. For temporary alinement work, the scale may be supported by the field-fabricated rod of adjustable height clamped in a tripod (fig. 15). This device is used not only for running lines in tunnels but also in surveys on city streets to raise the line of sight above parked cars or the heads of pedestrians.

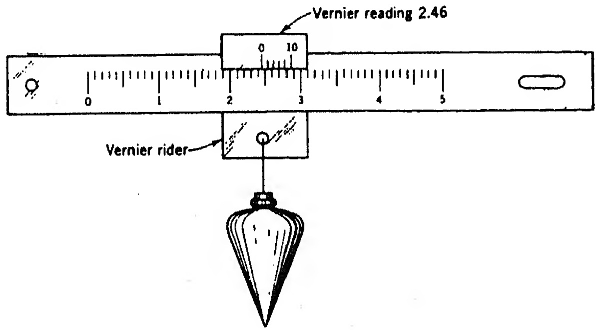


Figure 14. Brass scale.

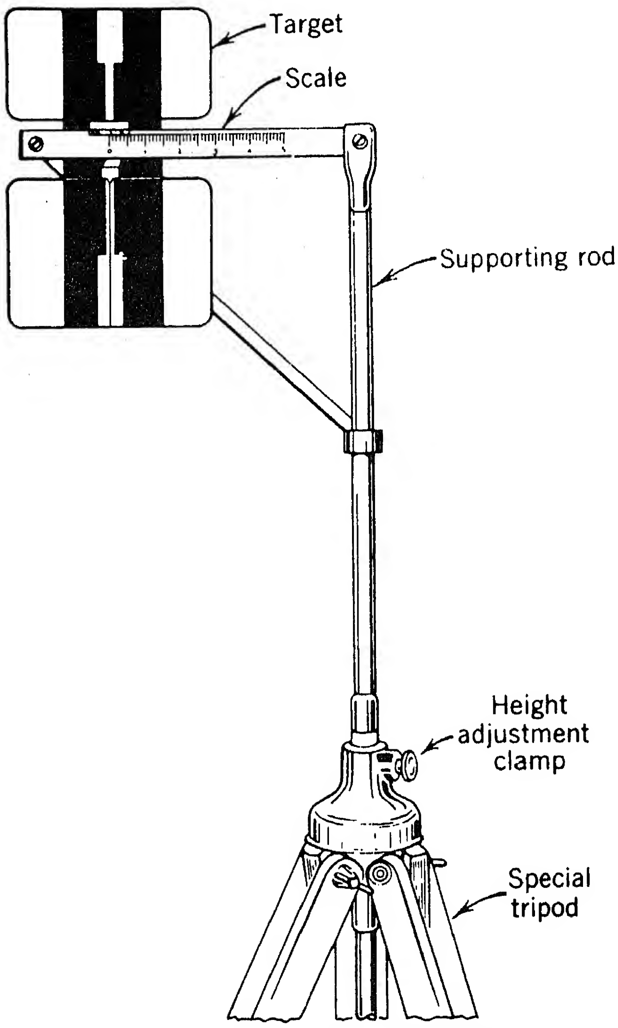


Figure 15. Brass scale target holder.

e. Signals. In the majority of underground surveys the same hand or voice signals as in comparable surface surveys (TM 5–232) are used for communication between the members of the survey party. An electric signal lamp held in each hand increases the visibility of hand signals. In shaft plumbing it is desirable to have telephone communication between personnel at the top and bottom of the shaft. Sound-powered telephones are ideal for this purpose, if available.

f. Shaft-Plumbing Devices. Where it is only necessary to plumb down a shaft occasionally, the simpler the device for holding the wire in position at the top of the shaft the better. A board or plank can be properly positioned at the top and the plumb bob wire hung over a notch cut in the edge of the plank or in the head of a spike driven into the plank, the excess wire being wound on a reel. Where repeated measurements are required, a somewhat more flexible arrangement for hanging the plumb bob is desirable. A bracket which permits shifting the wire a small amount in any direction is convenient. Where a line is to be carried underground by hanging two plumb bobs in the same shaft, the use of brackets of the type shown in figure 16 permits shifting the two wires into line or moving them closer together if necessary to prevent them from striking projections on the sides of the shaft. Figure 17 shows a somewhat different type of bracket with a reel for winding the wire.

g. Photographic Cameras. Much valuable information can be obtained during rapid reconnaissance surveys of caves and other underground passages by taking photographs to show the nature and extent of the rooms and openings. The 4- by 5-inch (10.0 x 12.7 cm) camera with flash attachment is an item of standard equipment and is well adapted to this purpose.

22. Care and Handling of Instruments

a. Underground Conditions. Surveying instruments are generally subject to more harmful conditions in underground work than in similar surveys on the surface. Many workings are damp and the instruments may soon become coated with a wet slime. Corrosive gases and acid fumes will tarnish graduations both on tapes and on instrument circles and verniers. Instruments made of aluminum are subject to severe corrosion in limestone mines

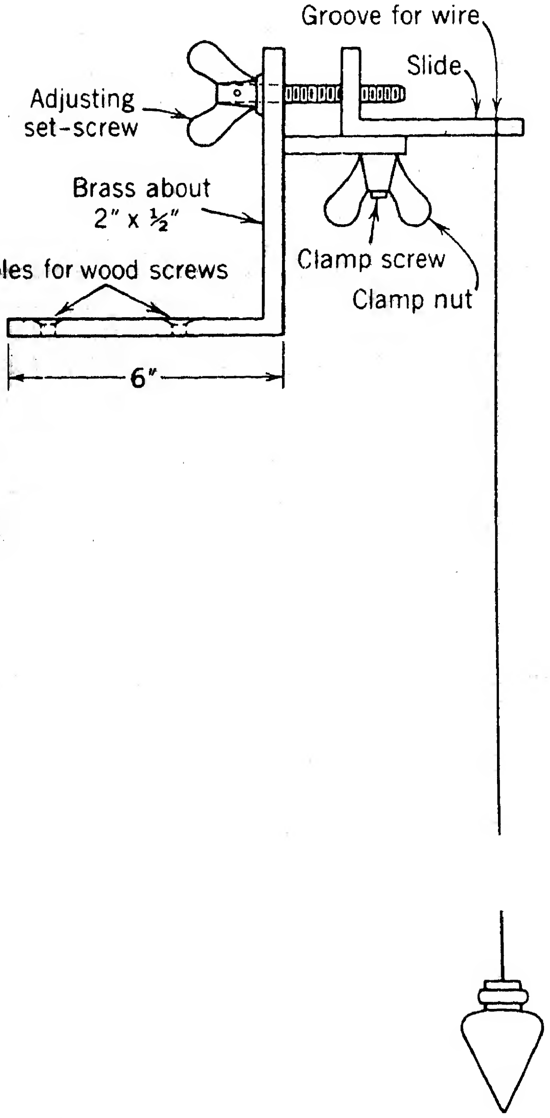


Figure 16. Shaft-plumbing bracket.

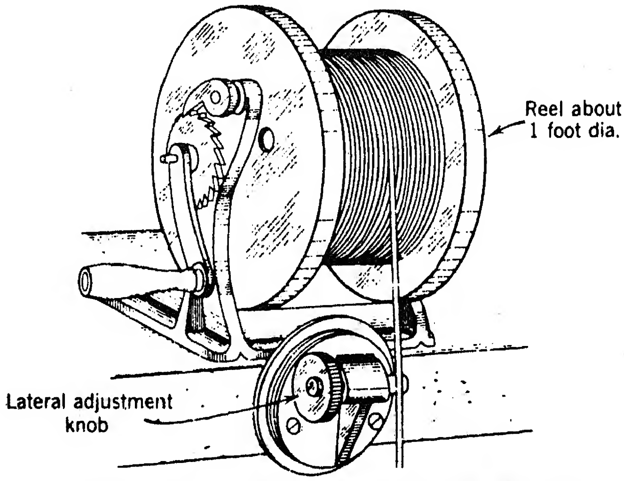


Figure 17. Shaft-plumbing bracket and reel.

or caves unless protected by special coatings. Cement and rock dust will work into leveling head screws, telescope slides, and other openings. Where the humidity is high, as is often the case underground, a drop in temperature may result in condensation on lens surfaces both inside and outside the telescope.

b. Instrument Selection and Care. Interior focusing instruments are more nearly moisture-proof and dustproof than those with an exterior focusing system, although both types will give satisfactory service in underground operations. The vertical circle should be protected with a full guard to minimize the effect of dripping water on the graduations. The instrument should be covered when not in actual use. Small bags of anhydrous calcium chloride or of silica gel are frequently placed in instrument boxes to serve as dehumidifiers. Lacking these, newspaper or blotting paper placed in the box when the instrument is put away at the close of each day's work will absorb much moisture. Dust on the objective and eyepiece lenses can be removed with a soft camel's-hair brush. Condensation on exterior lens surfaces can be removed with a clean chamois or lens tissue. When interior lens surfaces fog, it is best to take the instrument into a warm, dry room and remove the eyepiece until the condensation disappears, keeping the telescope out of strong drafts which might damage the crosshairs. Dust on graduated circles can be removed with a soft cloth or camel's-hair brush, stroking across rather than along the graduations and carefully avoiding touching the edge of the graduations. Tarnish can be removed temporarily with an artgum eraser, but cleaning can be done best with mild soap and water. Many recent instruments have circles of nontarnishing duralumin. Leveling screws will gum up and bind unless cleaned frequently. All cleaning of lenses, circles, and verniers and all field repairs of instruments should be entrusted to a qualified survey-instrument repairman assigned to support the survey party. Other survey personnel should not attempt instrument disassembly. Instru-

ments should be shipped to a repair depot for major repairs.

c. Importance of Instrument Adjustments. Because of the relatively short sights and large vertical angles so often observed in underground work, and because of the impracticability of making many of the check measurements which are possible in surface surveys, it is important that the instruments be kept in fine adjustment at all times. The majority of surveying operations can be carried out in such a manner that lack of adjustment will not affect the results, and while it is always good practice to follow such procedures, however, the work can be done more quickly and with greater assurance of accuracy if the instruments used are maintained in good adjustment. See TM 5–232 for the adjustment of instrument.

d. Care in Making Instrument Setups. Many underground sights are short. Angular measurements will be seriously in error unless the transit is set up precisely under the roof station or directly over a station in the floor. The care which is necessary in this operation cannot be overemphasized where accurate measurements are required. In tunnel work in particular, the tunnel alinement may be carried several kilometers from a very short base plumbed down a shaft. The instrumental work must be painstakingly performed and repeated time and again to assure the requisite accuracy.

Section III. LEVELING

23. Methods

The elevations of points in underground workings may be obtained by spirit leveling, trigonometric leveling, barometric leveling, vertical taping, or slope taping. Spirit leveling is particularly adapted for running levels from the surface into portals and for determining differences in elevation of points in tunnels, drifts, cross-cuts, and other underground passages which are substantially horizontal. Vertical taping is used in transferring elevations down vertical shafts. Slope taping and vertical angles are widely used for determining differences in elevation in either inclined or horizontal passageways. The other methods are employed on reconnaissance surveys or for special purposes. All of these methods are used in surface surveys (TM 5–232). In applying the methods to underground work, there are certain differences in procedure which are covered in paragraphs 24 through 28.

24. Spirit Leveling

The chief difference between spirit leveling underground and on the surface is that many of the underground turning points and benchmarks are located on points set in the roof of the passageway and hence are above the instrument. The rod is held on these points in an inverted position. Readings taken on an inverted rod are usually shown in the notes with a minus sign to indicate that they should be applied in the opposite manner to that normally used. A backsight read on an inverted rod must be subtracted from the elevation of the point to obtain the height of instrument, while a foresight so taken must be added to the height of instrument to obtain the elevation of the point. Figure 18 shows successive positions of the level and rod in running levels into a tunnel portal. Backsight and foresight readings are as indicated. Sample notes for this line of levels are given in figure 19. The usual arithmetic check is applied by taking the algebraic sums of the backsights and foresights. The computed difference in elevation should equal the algebraic difference of these summations.

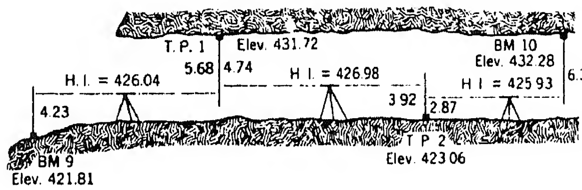


Figure 18. Underground leveling.

Leveling ~ West Portal ~ Great Notch Tunnel ~ 21 Aug. 19__					
Point	BS	HI	FS	Elev.	Remarks:
					⊼ & N Cpl. J.D.R., Rod. Pfc. T.M. Sands
BM9	4.23	426.04		421.81	Bolt in ledge 50' N.W. of portal
TP1	-4.74	426.98	-5.68	431.72	
TP2	2.87	425.93	3.92	423.06	
BM10			-6.35	432.28	Bolt in roof at Sta. 6+27
	+2.36		-8.11		
	-8.11				
	10.47	← Check	→	10.47	

Figure 19. Sample notes for line of levels shown in figure 23.

25. Trigonometric Leveling

The term trigonometric leveling, as used in such surface surveys as planetable operations and triangulation work (TM 5–232 and TM 5–441), applies to the determination of the horizontal distance and the vertical angle from a point of known elevation to a point the elevation of which is to be determined. From these data the difference in elevation of the two points is calculated, making appropriate corrections for curvature and refraction where the horizontal distance is great. Horizontal distances are seldom measured in sloping underground passages so the method has limited application in underground surveys. One such application, in determining the elevations of points for underground topography, is described in paragraph 61. The slope-taping method of leveling utilizes trigonometric computations but is classed as a separate method since the slope distance rather than the horizontal distance is used in the computations.

26. Barometric Leveling

Approximate elevations in underground reconnaissance surveys are obtained by baro-

metric leveling (TM 5–441). Formulas and tables for barometic leveling (TM 5–236) provide for an air-temperature correction but disregard terms which depend on humidity, latitude, variations in air constituents, and other factors. In caves and underground passages both the humidity and the CO_2 content of the air may vary considerably from conditions on the surface. The use of the aneroid barometer (or surveying altimeter) should be restricted to rough measurements where a determination of elevations to the nearest 6 meters (20 ft) will do. When the barometer is taken underground, ample time should be allowed for the instrument to adjust itself to the underground temperature before readings are taken. Twenty minutes should be enough.

27. Vertical Taping

Elevations are commonly carried down shafts by taping vertically from one level to another. Several variations in method are used.

a. Taping on Shaft Timbering. In this case, vertical measurements are made between points set in the shaft timbering 100 feet (30 meters) or less apart. One chainman stands on the top of the cage used to transport men or materials up or down the shaft. A second chain man sits on a seat secured to the hoisting cable at a point about 100 feet (30 meters) above the cage. The cage is lowered for each tape length in accordance with signals given by the head chainman or party chief. For precise work, corrections are made for temperature and standardization of the tape.

b. Tape Used as a Level Rod. Figure 20 shows a 300-foot steel tape hung in a vertical shaft. Assume the following data: tape is of standard length when fully supported at a temperature of 70° F. and under a tension of 20 pounds (9 kg); weight of tape is 6 pounds (2.7 kg); temperature in shaft is 90° F. at time of measurement; coefficient of expansion is 0.0000065 per ° F. A weight of 17 pounds (7.65 kg) is hung on the lower end of the tape. The upper end, which carries both the weight of the tape and the attached weight is then under a tension of 23 pounds (10.35 kg). The lower end is under a 17-pound (7.65 kg) tension. The average tension, over the portion of the tape used as a level rod, is substantially 20 pounds (9.0 kg). A level

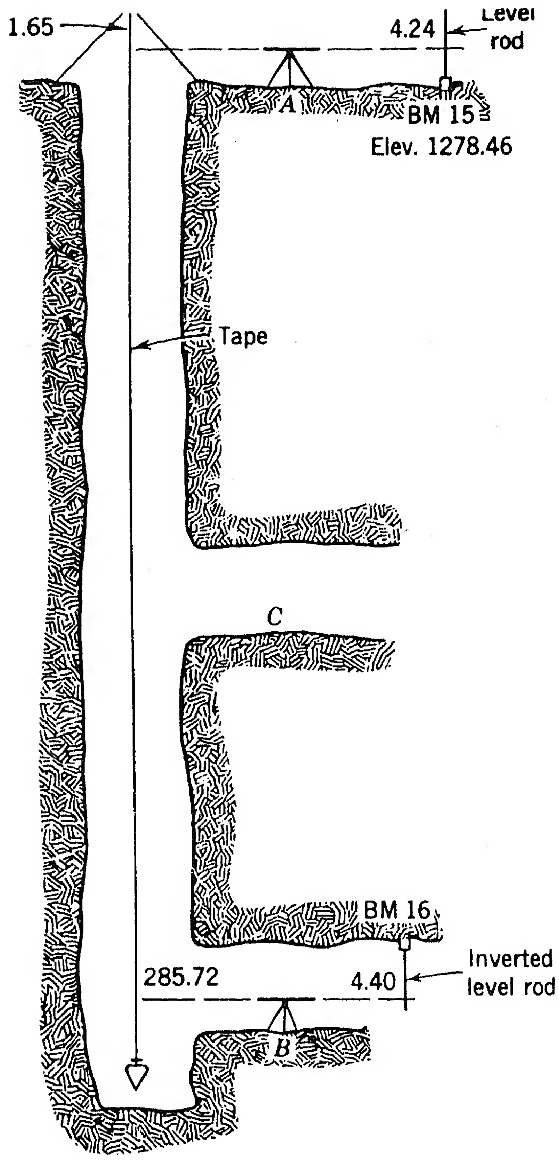


Figure 20. Tape used as a level rod.

is set up on the surface at *A* and a backsight of 4.24 feet is taken on *BM* 15 which is at elevation 1278.46 feet. A foresight of —1.65 feet is read on the tape. A second level at *B* takes a backsight of —285.72 feet on the tape and a foresight of —4.40 feet on *BM* 16, a point in the roof of the lower level. The elevation of *BM* 16 is required. The portion of the tape used as a level rod has a nominal length of 285.72 — 1.65 = 284.07 feet. Since the temperature at the time of measurement is 20° F. above that of standardization, a correction of $284 \times 20 \times 0.0000065$ = 0.04 feet must be added, making the actual distance between the lines of sight of the two

levels 284.11 feet. The elevation of *BM* 16 is then 1278.46 + 4.24 — 284.11 + 4.40 = 1002.99 feet. Elevations may be transferred to the intermediate level *C* in the same manner. For precise results, the weight hung on the tape should be reduced in this latter case so that the tension in that portion of the tape between *A* and *C* will average 20 pounds (9.0 kg).

c. Measurement of Plumb-Bob Wire. In this case, a heavy bob is hung in the shaft, the wire passing from a reel over a pulley at the top of the shaft. Two stakes, *A* and *B* (fig. 21) are set on the surface, under the wire, 100 feet (30 meters) apart. The relative elevations of the top of the plumb bob *C* and the underground bench mark *D* are determined by leveling. A marker is then fastened to the wire at *B* and the wire reeled in until the marker reaches *A*, indicating that 100 feet (30 meters) of wire has been reeled in. A second marker is then fastened to the wire at *B* and the process continued, measuring off each 100 feet (30 meters) and the final fractional length until the bob reaches the surface and the elevation of its top can be referred to a bench mark of known elevation. In this case, the tension at the bottom of the wire remains constant at the weight of the plumb bob. The tension at the top varies from the sum of the bob weight plus the weight of the whole length of wire to the weight of the bob alone as the wire is slowly reeled in. The mathematical expression for the tension correction is not a simple one. Tension is usually ignored and the method is not used for precise determinations of elevation. A further complication arises from the fact that the length of a 100-foot (30 meter) section of the wire, when stretched from *A* to *B*, is not necessarily of the same length which it had when hung in the shaft, because the temperature of the section of wire at the surface may be considerably different from the temperature it had when hung in the shaft. The method is therefore restricted in application to the rapid determination of approximate elevations (to plus or minus ½ ft or 15 cm) in deep shafts. The method in *a* above is capable of yielding precise results but is time-consuming. The method in *b* above will give a close determination but is limited to use in shafts of less depth than the length of the longest tape available (usually 300 ft or 90 meters).

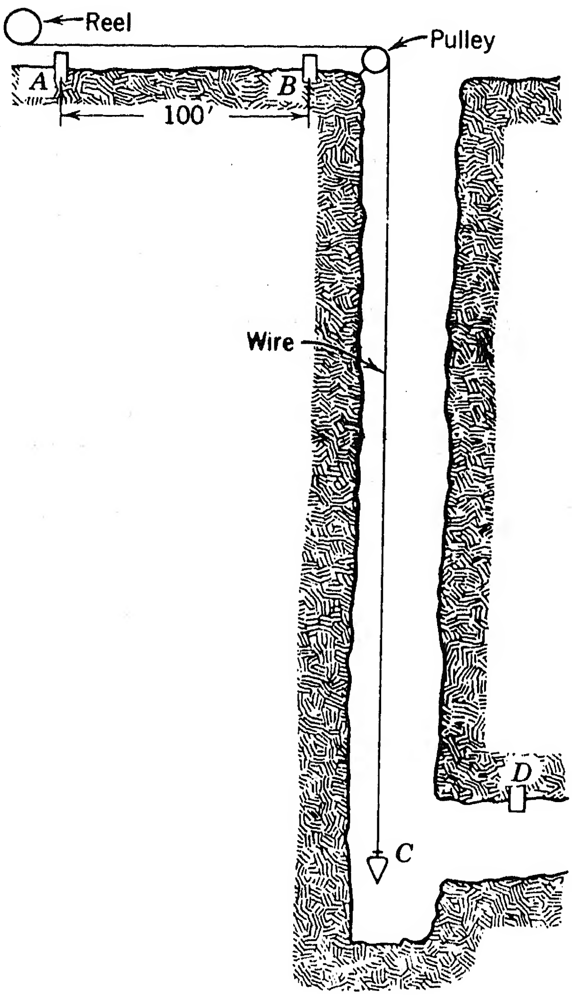


Figure 21. Elevations from measurement of length of the plumb-bob wire.

28. Slope Taping

Slope-taping and vertical-angle measurements (fig. 22 are widely used for determining differences of elevation in underground surveys, particularly in sloping passageways. The elevation of point *A* has been previously determined to be 376.42 feet. The transit is set up below this point and the height of instrument (*hi*), the distance from the spad at *A* to the horizontal axis of the telescope, is measured with a pocket tape. It is usually convenient to measure from the spad to one end of the horizontal axis. This is accurate enough unless the *hi* is very small, in which case a correction should be made to obtain the true distance to the center of the telescope. The *hi* shown in

figure 22 is 3.78 feet. A plumb bob is then hung from the spad at B and the height of sight (*hs*); the distance from the spad to the top of the plumb bob, is measured. In this case *hs* is 2.96 feet. The vertical angle from the transit at C to the top of the plumb bob at E is measured (—6° 10′ in this instance) and the slope distance from the center of the telescope to the top of the bob is taped (176.40 feet). In such taping, the tape and spring balance are turned into a vertical plane so that both tape and balance can be read when holding the tape above the transit telescope. For precise work the slope distance is corrected for standardization, temperature, and sag. Tension corrections are not required if the field tension is the same as that of standardization. Assuming the above value to be correct slope distance, $DE = 176.40 \sin 6° 10' = 18.95$ feet. The elevation of point B is therefore $376.42 - 3.78 - 18.95 + 2.96 = 356.65$ feet. For notekeeping purposes, upward vertical angles are considered positive and downward angles negative; the *hi* is positive if the instrument is above the station, negative if it is below it; the *hs* is positive if the point sighted is below a station, negative if it is above it. Rules can be established for determining the elevation of a point by adding or subtracting the *hi*, *hs*, and sine distance from the known elevation. The surveyor not constantly engaged in underground surveys may not remember these rules and a rough sketch similar to figure 22 may be drawn to show the correct relationship between the several distances which have been measured or computed. The horizontal distance CD between A and B is determined at the same time as the difference in elevation, using either the cosine of the vertical angle to obtain the distance directly, or using the versed sine relationship to compute a correction to the slope distance. On reconnaissance surveys, the vertical angles are often measured by clinometer, pocket transit (para. 17), or Abney hand level (TM 5–232) rather than with the engineer transit.

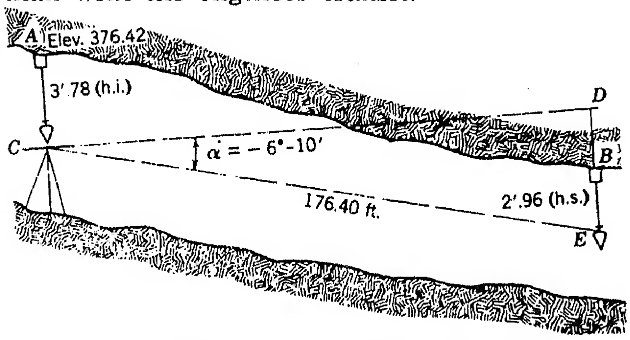


Figure 22. Difference in elevation by slope taping.

Section IV. DISTANCE MEASURING

29. Taping

The taping party normally is a subparty of a traversing party, and its primary duty is to make the taped measurements necessary to determine the horizontal distances between points. The minimum number of men required for a taping party depends on the taping methods and equipment to be used, which in turn depends on the accuracy to be attained. For methods see TM 5–232.

30. Electronic

Although taping is the best method for measuring short distances at present, the electronic distance measuring devices are the most accurate over a long distance. Again the accuracy to be attained will determine the equipment to be used. For electronic distance measuring see TM 5–232.

Section V. ANGLES AND AZIMUTH

31. Measurement of Horizontal Angles

In underground reconnaissance surveys, horizontal angles are frequently determined from bearings taken with a pocket compass, a hanging compass, or a Brunton pocket transit (para. 17). Precise angular measurements are made with a transit or theodolite. Angles are always turned in a clockwise direction. Where the lines of sight are not greatly inclined, the sights are made through the main transit telescope and the horizontal angles are measured exactly as in surface surveys (TM

5–232). When one or both of the sights taken are steeply inclined, an auxiliary telescope or a prismatic telescope or eyepiece is used in making the steeply inclined sight or sights. An auxiliary telescope may be mounted either above or to one side of the main telescope. One sight may be made through the main telescope while the other is taken through the auxiliary telescope, or it may be necessary to take both sights through the auxiliary telescope or through a prismatic telescope.

32. Horizontal Angles With Top Telescope

When the auxiliary telescope is mounted above the main telescope and properly adjusted so that the two telescopes swing in the same vertical plane and the two lines of sight are parallel, a horizontal angle obtained by sighting through the top telescope will be identical with that which would have been obtained had it been possible to sight through the main telescope. When a single measurement of an angle is desired, there is a distinct advantage in the use of a top telescope rather than in using one mounted on the side. On the other hand, to increase accuracy and to eliminate instrumental errors, it is common practice to at least double all horizontal angles, inverting the telescope between the first and second measurements. A transit cannot be plunged with the top telescope in position. The auxiliary telescope may be mounted on the side and used in such a way that no correction need be applied to one-half of a doubled angle.

33. Horizontal Angles With Side Telescope

a. Main Telescope Used for One Sight, Side Telescope Used for the Other. When one of the two sight lines is nearly level and the other is steeply inclined, the first sight may be taken through the main telescope while the side telescope is used for sighting the other. The condition for a single measurement of the angle is shown in figure 23. The backsight on A was taken through the main telescope, the foresight on B was observed through the side telescope. The angle through which the horizontal circle has been turned is equal to angle AIC whereas the desired angle is angle AIB. The measured angle must be increased in this case by the correction angle α, the tangent of which is equal to the eccentricity, e, of the side telescope divided by the horizontal distance d between the instrument and point B. This horizontal distance is obtained by multiplying the taped slope distance from the instrument to B by the cosine of the vertical angle, or by multiplying the slope distance by the versed sine of the vertical angle and subtracting this correction from the slope distance.

b. Both Sights Taken Through Side Telescope. Figure 24 shows the condition for a single measurement of a horizontal angle (AIB) when both sights are observed through the side telescope. The angle through which the horizontal circle has been turned is angle CID. The angle sought is angle AIB. This equals CID minus α plus β. Values of α and β are computed as in the preceding case from the eccentricity of e and the two horizontal distances d_1 and d_2. The reading of a single angle must be reduced to center by correctly applying the values of α and β except in the case where d_1 and d_2 are equal. In this case α and β will be equal and one correction will exactly balance the other.

c. Method for Reading Angles Without Corrections. In figure 23 the actual angle turned on the horizontal circle equals AIC. This is equal to the correct angle AIB minus α. If the

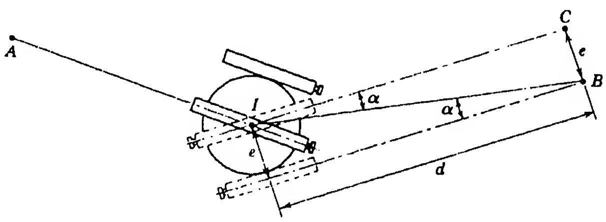


Figure 23. Correction to horizontal angle for single side-telescope sight.

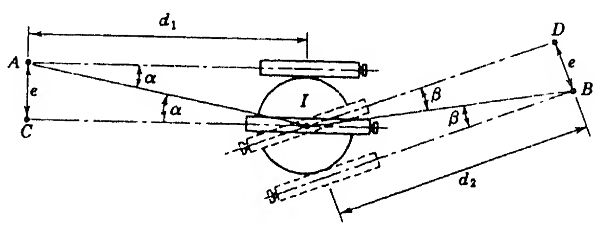


Figure 24. Correction to horizontal angle for side-telescope sights.

instrument is now plunged and again sighted on A, the auxiliary telescope will be on the left of the main telescope instead of on the right. This condition is shown in figure 25. Then, if a second angle is added to the first by again sighting B through the auxiliary telescope, the second angle turned will equal AIC' (fig. 25). This equals the true angle AIB plus α. Since the two angles turned have been added together on the circle the following applies:

Final reading $= AIC + AIC' = AIB - \alpha + AIB + \alpha = 2AIB$

One-half of the final reading gives the correct value of the angle and no correction is required. Similar reasoning applies to the case shown in figure 24. While the correct angle can be obtained from two repetitions, there is not the check normally obtained when an angle is doubled since the first reading will not be one-half of the second. There is no point in reading or recording the first angle. When a check is desired, four repetitions should be made, reversing the telescope between each measurement, and reading and recording the second and fourth angles. One-half of the final reading should then equal the second angle and one-quarter of the final reading will give the correct value of the angle measured.

34. Horizontal Angles With Prismatic Telescope

a. Prism on Main Objective. Before the prism is attached, a point is sighted at a considerable angle above or below the instrument and is bisected with the vertical crosshair. The prism is then attached and the point again brought into view, the prism being turned by means of its tangent screw until the point is again bisected by the vertical crosshair. Horizontal angles are then read in the normal manner.

b. Prismatic Eyepiece. When a steep upward sight is made through a prismatic eyepiece

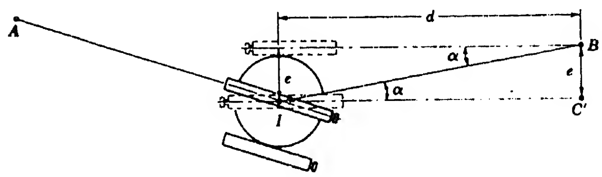


Figure 25. Correction to horizontal angle for single side-telescope sight (auxiliary telescope on left).

there is no correction to be applied to a measured horizontal angle.

c. Prism on Side Objective. Prismatic telescopes of this type give an eccentric line of sight on horizontal angles just as is the case with a side telescope. Single angle measurements are corected as indicated in paragraph 33. Corrections are eliminated and accuracy increased by repetitions with the instrument in the direct and reversed positions.

35. Measurement of Vertical Angles

Approximate values of vertical angles can be obtained with the service clinometer, hanging clinometer, or Brunton pocket transit (para. 17). Precise measurements are made with the transit, or theodolite, following procedures used on surface surveys, except where the sights are steeply inclined. Vertical-angle measurements with auxiliary or prismatic telescopes are discussed in the following paragraphs.

36. Vertical Angles With Side Telescope

The side telescope is positioned and adjusted so that, with both telescopes level, they are in the same horizontal plane and their lines of sight are parallel. Under these conditions, the vernier of the vertical circle should read zero. A sight can then be taken through the side telescope and the vertical angle read directly on the vertical circle without any correction. A reading should also be taken with the telescope reversed and the mean of the two used, thus eliminating any index error. If the transit has two verniers on the vertical circle, both should be read to eliminate possible eccentricity of the circle.

37. Vertical Angles With Prism on Side Objective

This type of prismatic telescope has the same characteristics, insofar as vertical angle measurements are concerned, as a transit with a side telescope. Vertical angles are correctly read directly on the vertical circle.

38. Vertical Angles With Prismatic Eyepiece

Where a prismatic eyepiece is used on the main telescope, vertical angles are read directly on the vertical circle without correction.

39. Vertical Angles With Top Telescope

For the steeply inclined sight shown in figure 26 with point B sighted through the top telescope, the actual angle through which the vertical circle has been turned below the horizontal is angle HIC. The desired angle is HIB. The measured angle must therefore be reduced by the amount of the correction angle α to obtain the correct angle. The sine of α is equal to the eccentricity, e of the the top telescope divided by the slope distance IB from the instrument to the point sighted. This slope distance is normally measured by taping. Since the eccentricity of the top telescope is constant for a given transit, a table can be prepared giving values of α for various values of the slope distance IB. This table will make possible the rapid reduction of vertical angles to center. Since a vertical angle cannot be measured by the method of repetition, it is not possible to eliminate the correction as could be done in measuring a horizontal angle with a side telescope. The correction is much more readily determined than the correction for a horizontal angle observed through a side telescope because the slope distance is used directly instead of having to first compute the horizontal distance.

40. Vertical Angles With Prism on Main Objective

Figure 27 shows a steeply inclined sight observed through a telescope having a 90° prism on the main objective. The desired vertical

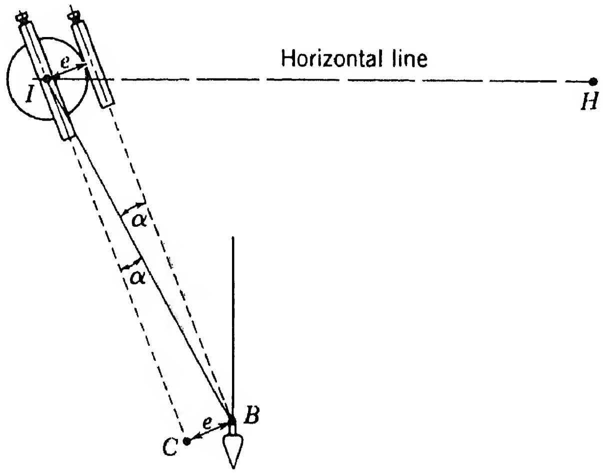


Figure 26. Vertical-angle correction for top telescope.

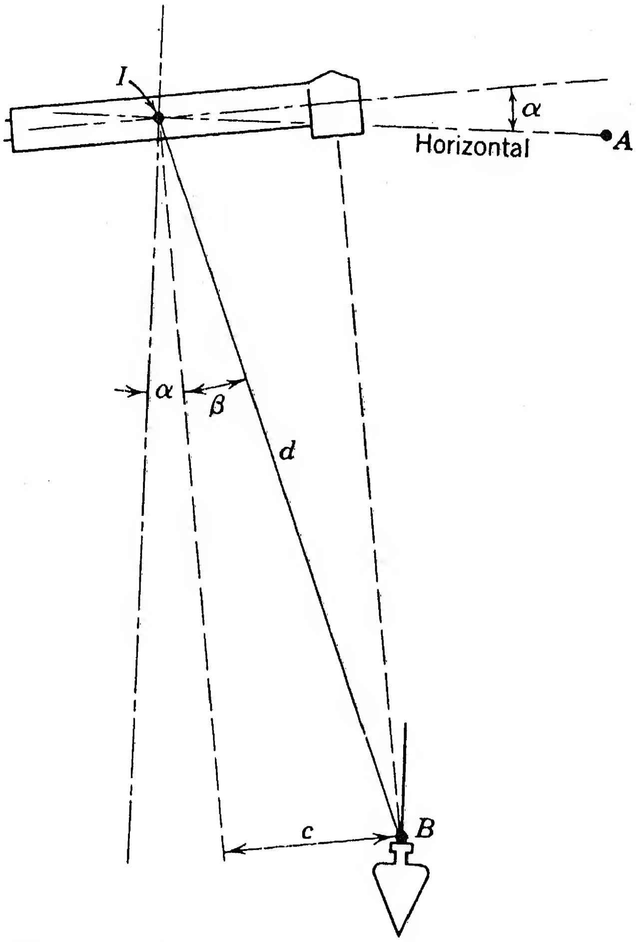


Figure 27. Correction to vertical angle measured with prism on main objective.

angle is AIB (negative). The actual angle read on the vertical circle is α (positive in this case). The sine of angle β is equal to c, (the distance from the center of the instrument to the prism), divided by d, (the measured slope distance from I to B). The angle sought, AIB, equals $90° - \alpha - \beta$. For a given instrument, values of β can be tabulated for various values of the slope distance d, thus permitting a rapid reduction to center.

41. Choice of Position for Auxiliary Telescope

On the majority of mining transits the auxiliary telescope can be placed interchangeably in either the top or side position. It might appear desirable to measure all horizontal angles with the top telescope and all vertical angles with the side telescope so that no corrections would be required. It is not practicable,

however, to change the position of the telescope between horizontal- and vertical-angle measurements at any one setup, because of the time required to get the two telescopes in the same plane with their lines of sight parallel. The preferred procedure is to use the side telescope since no corrections are required on vertical angles, and horizontal-angle corrections can be eliminated if the angles are measured by repetition with the telescope alternately in the direct and reversed positions.

Section VI. SHAFT-PLUMBING AND TRANSFER OF AZIMUTH

42. Wires and Plumb Bobs for Shaft-Plumbing

The azimuth of a line in an underground passage is frequently established by plumbing down a shaft from located surface points. Heavy plumb bobs are used for this purpose.

a. Plumb Bobs and Dampeners. The weight of the plumb bob which is necessary to keep the suspending wire vertical will depend upon the depth of the shaft, the strength of air currents within the shaft, the presence or absence of dripping water in the shaft, and the size of wire used. On military construction, the conditions to be met can be observed during the shaft sinking and various weights can be tested. Similar tests can be conducted in established shafts before ordering a special plumb bob, making use of any convenient heavy weights in the tests. In any case, these heavy plumb bobs and dampeners (para. 21*b*) are not items of military issue and must be improvised or procured from other sources. The weight of the plumb bob should be approximately equal to one-third of the breaking load (*b* below) of the wire used.

b. Wire Material and Size. Military stocks include the following kinds and sizes of wire which can be adapted for this purpose: steel wire, annealed, 0.0348″, 0.0475″, 0.0625″, 0.0800″; steel wire, zinc-coated, 0.0348″, 0.0625″, 0.0800″; high-carbon, spring, music wire, 0.030″, 0.045″, 0.063″, 0.095″. These steel wires must be carefully handled to avoid kinking. The music wire has a characteristic springiness and is likely to become badly snarled and tangled if the weight is suddenly removed. The zinc-coated wire, because of its greater resistance to corrosion, is preferred for repeated measurements in a wet shaft. The annealed wire should be kept oiled to minimize rusting. Since the plumb-bob wire is often used as a backsight for a transit at the bottom of the shaft, and since the sight is usually short, the diameter of the wire should be small. Where the air currents in the shaft are strong, the weight of the plumb bob must be increased and the larger-sized wires used to carry this increased weight. Under these conditions, brass scales (para. 21*d*) should be used for instrument alinement. Data on steel wire and appropriate plumb-bob weights are given in table I. The weight of the wire itself is small and will not affect the size of plumb bob used except in unusually deep shafts. The buoyant effect of the oil in the dampening drum can be disregarded in selecting the weight of the plumb bob.

c. Apparatus for Supporting Wires. See paragraph 21*f*.

Table I. Steel Wire and Plumb Bobs for Shaft-Plumbing

Wire size (inches)	Approximate breaking load (pounds)	Weight of wire (pounds per 1,000 feet)	Approximate weight of plumb bob (pounds)
0.0348	95	2.7	30
0.0475	175	5.8	60
0.0625	310	10.5	100
0.0800	500	16.9	150

43. Precautions and Procedures in Shaft-Plumbing

a. Position of Wire Supports. Where only one wire is to be hung in a shaft, it should be hung from a point near the center of the shaft opening. Where two wires are to be hung in the same shaft, they should be placed as far apart as possible to obtain the maximum length of base line at the underground level. However, positions in opposite corners of a rectangular shaft lead to unsatisfactory results where strong air currents are present.

Air velocities in excess of 1.5 to 2 feet per second (1 to $1\frac{1}{2}$ mi per hr) will cause deflection of the plumb wires. A wire hung in the corner of a rectangular shaft will be deflected in the direction of the long side. Wires hung in diagonally opposite corners will be deflected in opposite directions and the azimuth of the base at the bottom of the shaft may be appreciably different from that of the line between the wires at the surface. More satisfactory results are obtained when the wires are hung at the midpoints of the short sides of the shaft.

b. Reduction in Effect of Air Currents. During shaft-plumbing, forced draft ventilation systems should be turned off if practicable so as to reduce the velocity of air currents in the shaft. In many instances, all workings leading from the shaft have been temporarily sealed off to reduce deflection of the plumb wires. Where such steps cannot be carried out, wooden flues are often constructed from just above the bottom to the top of the shaft and the plumb wires lowered through them. This procedure is also followed in wet shafts to prevent disturbance of the wire by falling water.

c. Lowering of Wire in Shaft. A relatively light plumb bob (16-ounce) is customarily used to carry the wire down the shaft. This will minimize the danger of serious accident should the wire break. A snap hook on the lower end of the wire will permit removal of the light bob and attachment of the heavy bob when the wire is in place.

d. Avoidance of Contact of Wire With Shaft. It is essential that the wire hang freely from its point of support. Several methods are available for checking this. The bob may be swung as a pendulum and the observed period compared with that computed for a pendulum having a length equal to that of the wire. The point of support at the top may be moved through a measured distance and this distance compared with the movement of the bob at the bottom. A light, slowly revolved around the bob and viewed from the top of the shaft, will often show if the wire is touching a projection of the shaft wall. A small ring, wrapped loosely around the wire, should drop freely from top to bottom if the wire is unobstructed.

e. Use of Dampeners. The swing of the plumb bob must be properly dampened as described in paragraph 21*b*.

f. Use of Three Wires. Three wires are often hung in line in a single shaft to detect any displacement of the bobs resulting from air currents, falling water, or obstruction. If the three wires are found to be at the same distance apart at the bottom as at the top, it is reasonable to assume that there is no displacement, since it is unlikely that all three would be displaced in the same direction and by the same amount.

g. Signaling Devices. See paragraph 21*e*.

44. Transfer of Azimuth by Plumbing Down a Single Vertical Shaft

a. General Problem. The method described in this paragraph applies to the situation where only one vertical shaft leads from the surface to the underground passageways. It also applies to tunnel construction where, although there may be a number of shafts sunk to the tunnel grade, there is no opportunity to connect them by underground traverse until the tunnel has been holed through. Thus the headings extended from each shaft must be controlled in alinement from a very short base line plumbed down from the surface. Care must be used in establishing the correct alinement of this base since any error will be multiplied many times in the length of the tunnel. The discussion which follows describes the process of shaft-plumbing in establishing the line of a tunnel, using for an example, the headings projected from shaft *E* of figure 1. The principles used in transferring azimuth to a cave or mine are essentially similar. In such cases the extreme care and high precision required in tunnel work may not be necessary.

b. Preliminary Operations. Run out the line of the tunnel on the surface as described in paragraph 10. Set permanent points on line near the top of the shaft so that the alinement can be projected onto the shaft collar whenever necessary. At least three such monuments should be set and carefully referenced so that

any disturbance by construction operations can be detected and corrected. When the shaft has been excavated to the tunnel grade, hang plumb wires on line close to the shaft walls. Insert plugs in holes drilled in the shaft walls behind each wire just above the level of the tunnel roof. Each plug is to be made long enough to nearly touch a wire so that the mean position of each wire can be readily transferred to the plug and marked by a spad. Hang small plumb bobs from these spads and use these for sights to aline, by eye, the first 100 feet (30 m) or so of excavation in the tunnel headings. As indicated in figure 28, these operations will carry the construction far enough to permit setting up a transit in the tunnel at a little distance from the shaft so that an accurate alinement can be established for the extension of the headings.

c. Observations at Top of Shaft. For the accurate alinement of a tunnel, a transit is set up on the surface in the line of the previously established control monuments and the plumb-wire supports are adjusted until the wires are in line with the vertical crosshair of the transit. Two wires are necessary; a third is often hung to provide a check (para. 43). For surveys in caves, where an azimuth can be determined from any line transferred to the bottom of the shaft, it may prove simpler to fix the wires, jiggle the transit into the line with them (para. 46*a*), and then turn the angle from this line to a line of known azimuth. The micrometer head or lateral adjuster (para. 16*d*) is useful in alining the transit with the wires.

d. Determination of Mean Position of Wires. Where a shaft is of shallow depth and the wires are undisturbed by air currents or falling water, it is possible to jiggle a transit, set up in the heading near the bottom of the shaft, directly into line with the wires. When the vibration of the wires is too great for direct sighting, it is necessary to determine their mean position. The usual method of doing this is to fix a brass or translucent scale (para. 21*d*) close to each wire and near the roof of the tunnel. A transit is then set approximately in line with the wires and an extended series of observations made of the scale readings corresponding to the extreme positions of the wires. From these, the mean position is determined and the vernier rider and target is clamped on each scale in this mean position. Thus the line is fixed at the bottom of the shaft and it only remains to extend this line into the headings. This work is commonly performed during an off-shift period when construction operations are suspended, when forced-draft ventilation can be discontinued, and when there is no movement of material or personnel up or down the shaft. Several such periods may be required to thoroughly check the mean position of the wires and to project the line into the headings.

45. Transfer of Azimuth by Plumbing Down Two Vertical Shafts

There are often two or more vertical shafts leading from the surface to the underground workings. Where these shafts are connected by passageways at one or more levels, the transfer of azimuth from the surface to control the azimuth of underground surveys is less difficult than where only a single shaft is available.

a. Required Surface Surveys. Figure 29 shows a typical traverse for establishment of azimuth underground. Points *A* and *F* are

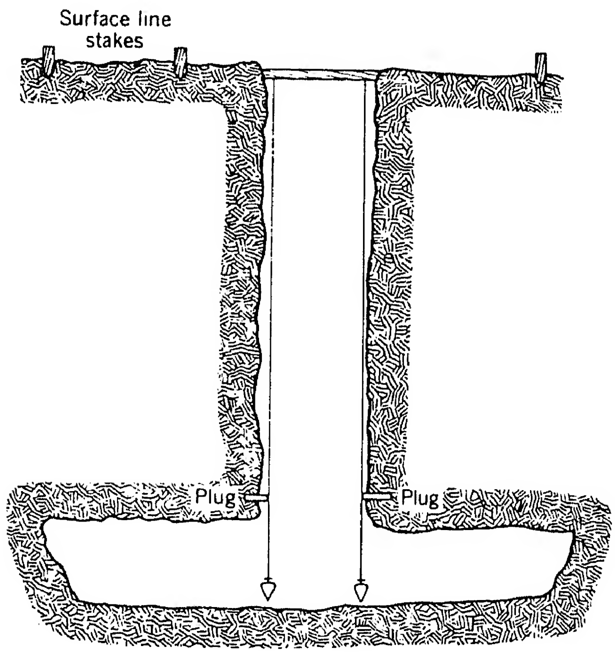


Figure 28. Preliminary alinement of shaft headings.

points established at the tops of the two shafts. Occasionally these are intervisible and the intervening terrain is suitable for taping. In this case, the distance *AF* is taped and the true bearing or azimuth of the line is determined by astronomical observations. As a rule, however, the length and bearing of this line must be determined by traversing or by triangulation *ABCDEF* (fig. 31) represents a surface traverse connecting the two shafts. The bearing of one side is established astronomically, angles are measured at *B*, *C*, *D*, and *E*, and all traverse sides from *A* to *F* are taped and checked. From these data the length and bearing of the closing side *FA* are computed (TM 5–232). To detect errors in the fieldwork, it is always advisable to physically close the surface traverse measuring angles and distances from *F* back to *A* along a different route than that used in the forward survey. The length and bearing of *FA* may also be established by triangulation, heavy timber or steel beams being secured over the tops of the shafts to serve as supports for the instrument and observer at these points.

b. Required Subsurface Surveys. Points *A* and *F* are then plumbed down their respective shafts, points *A′* and *F′* representing the positions of the wires at the underground level. A transit at *G′* is backsighted on the mean position of the wire at *A′* and an underground traverse *A′G′H′I′J′K′F′* is extended through the underground passageways to *F′*, all distances and angles being carefully measured and checked. A bearing is assumed for line *A′G′*, and the underground traverse is computed to determine the length and bearing of the closing side *F′A*. This length should check closely the length *FA* as computed from the surface traverse. Since the true bearings of these lines must be identical, the differences between the true bearing of *FA* and the bearing of *F′A′* as computed from the assumed bearing of *A′G′* is applied as a correction to the bearings of all lines of the underground traverse. Errors in the underground traverse can be detected and eliminated if this traverse, like the one on the surface, is physically closed by running back from *F′* to the starting point at *A*. It is likely that the same underground passages must be used for the return survey but new transit stations should be selected so that all angles and distances will differ from those measured on the forward run.

46. Extension of Line From Shaft Into Underground Passageway

Two methods, jiggling into line and triangulation, are in common use for extending the line established at the bottom of a shaft into

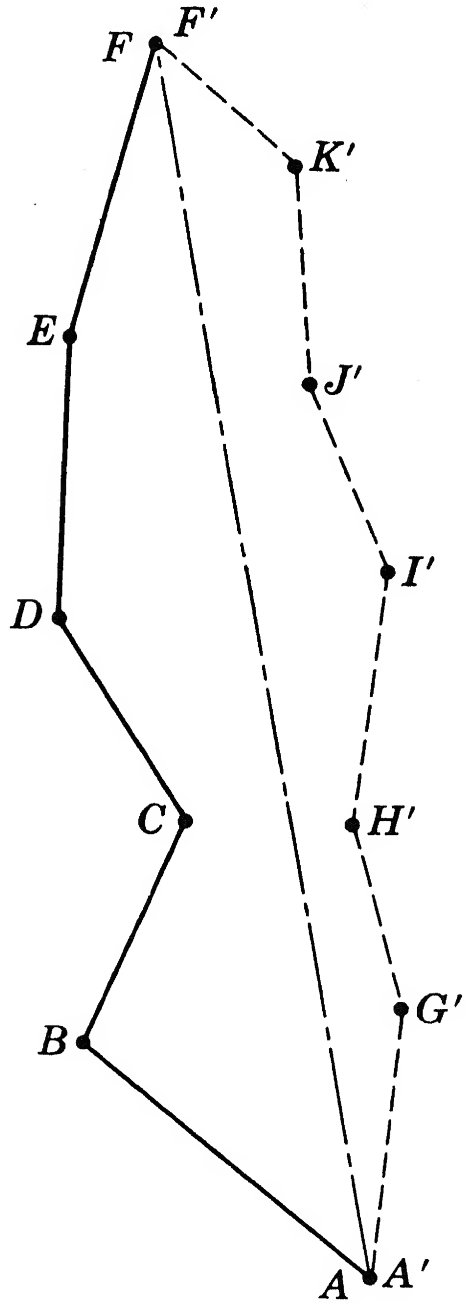


Figure 29. Traverse for establishment of azimuth underground.

a tunnel, or for determining the azimuth of a survey line in other underground passages.

a. Jiggling-into-Line Method. This method is also called coplaning. The transit is set up in the tunnel heading or other passageway at a short distance from the shaft and juggled into line with the plumb wires or with their mean positions as marked on brass scales. In this operation the instrument is set approximately on line by eye and its position is shifted slightly on successive trials until the line of sight, with the telescope in both direct and reversed positions, is in accurate alinement with the mean positions of the two wires. In perfecting the positioning of the instrument a micrometer head or lateral adjuster (para. 16*d*) is most useful but not essential.

(1) *Tunnel construction.* In tunnel construction, the transit is located on the axis of the tunnel. The telescope is plunged and the line projected onto three scales hung from the roof of the tunnel heading at intervals of 100 feet (30 m) or more. Readings are taken on each scale and the measurements are repeated a number of times with the instrument in both the direct and reversed positions to correctly establish and check the alinement. A vernier rider carrying a small plumb bob or target is then set at the mean reading on each scale and clamped in position. These plumb bobs are used to control the extension of the heading. The alinement is fixed by three scales rather than two so that any error can be detected should one of the scales be disturbed by construction operations in the tunnel. The line as fixed on the scales is projected forward onto other scales as the excavation of the tunnel is extended farther.

(2) *Surveys in other underground passageways.* In caves or other underground workings where the passage is not straight, the angle is turned from the transit (on an extension of the line between plumb wires) to the next transit station which has been established in the passage. This angle is measured by repetition (being doubled at least) with the telescope in both the direct and reversed positions to increase the accuracy and to eliminate instrumental errors. Figure 30 shows a typical situation using this operation. A transit at *C* is jiggled into line with two wires hung in the shaft at *A* and *B*. The distances *AB, AC, BC,* and *CD* are taped and

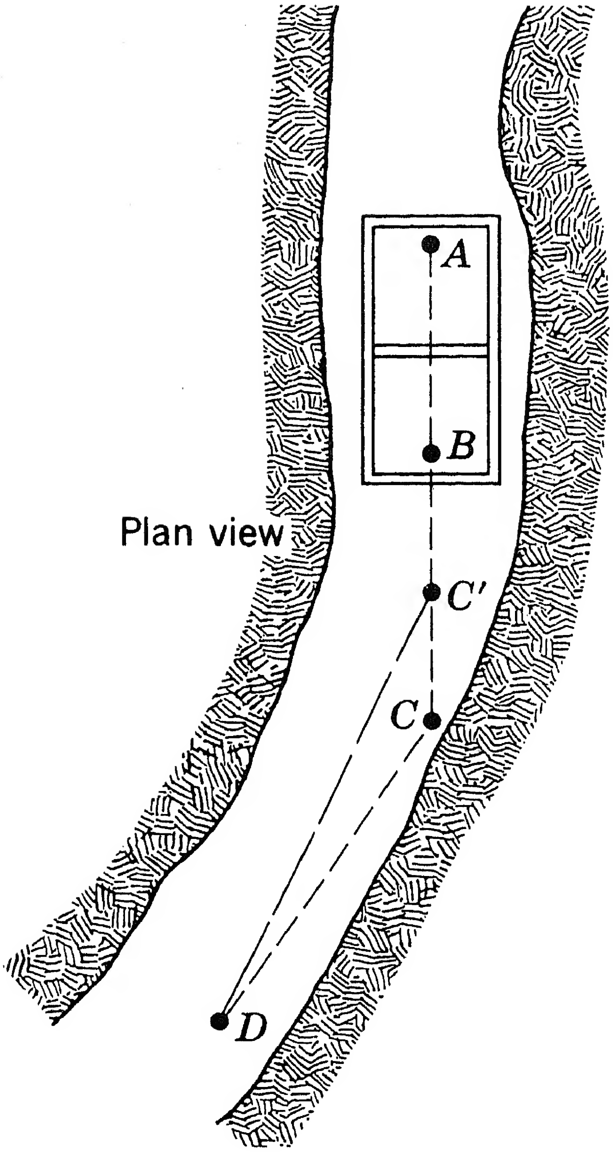


Figure 30. Determination of azimuth in an underground passage.

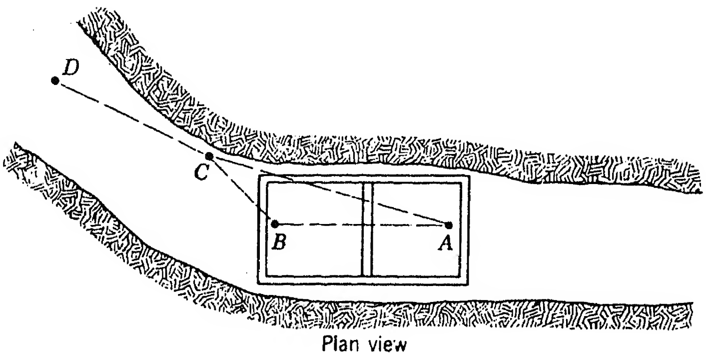


Figure 31. Establishing azimuth in an underground passage by triangulation.

the angle ACD is measured. The distance AB should equal the distance between the wires as measured at the surface. The azimuth of AB and the rectangular coordinates of points A and B are known from surface control surveys. This azimuth and the distance AC provide the azimuth and length of the first side of the underground traverse. From the measured horizontal angle at C, the azimuth of CD can be computed. The traverse is then extended beyond D, computing the true azimuth of each course in turn from the measured horizontal angles. The position of D can be checked by jiggling-in at C' and measuring corresponding distances and angles from this point.

b. Triangulation Method. This method is preferred by many engineers, particularly in situations where bends in the passage prevent the extension of a straight line for a long distance. In figure 31 A and B represent the two plumb wires hung in the shaft. The transit is set up at C, as close to B as proper focusing permits. A short-focus attachment (fig. 4) is useful. Distances AB, BC, AC, and CD are measured (D being the next station in the underground traverse). Angles BCD and ACB are measured by repetition. The distance AB should equal the distance measured between wires at the surface. In the triangle ABC, the three sides and the angle at C are known. The angles at A and B can be computed by applying the trigonometric law of sines. Since the azimuth of AB has been determined from surface surveys, sufficient data are available to compute the azimuth of CD.

47. Transfer of Azimuth by Transit

a. Into an Adit or Inclined Passageway. When an adit or inclined passageway leads into underground workings, the transfer of true azimuths to the underground survey lines is relatively simple. The transit is set up at the entrance and oriented on a surface line, the bearing of which has been previously established. The angle to an underground station in the adit is then measured by repetition, with the telescope in both the direct and reversed positions. Where the passageway is steeply inclined, a striding level should be used to make sure that the horizontal axis of the transit is truly horizontal.

b. Down a Shaft. In transferring azimuth down a steeply inclined or vertical shaft, a transit provided with an auxiliary or prismatic telescope can be used. The instrument is set up at the top of the shaft in such a position that the full width of the bottom of the shaft can be viewed through the telescope. The transit is sighted along a line of known bearing on the surface and then sighted down the shaft. A wire is stretched horizontally across the bottom of the shaft and adjusted in position until its entire visible length lies on the line of sight of the transit. Repeated observations are made with the instrument in both direct and reversed positions to check the alinement of the wire. This wire will then have the same bearing as that of the line on the surface. A transit is set up at the bottom of the shaft, jiggled into line with the wire, and then used

to set permanent reference points in the roof along this base line. A long straightedge can be used in place of the wire, or points on the transit line can be instrumentally set at the bottom of the shaft. The coordinates of one point at the bottom of a vertical shaft are established by plumbing. In an inclined shaft, the vertical angle and slope distance must be measured to a point at the bottom to furnish data for the computation of coordinates.

c. By Collimator. A collimator is an instrument widely used for centering a theodolite mounted on a triangulation tower over a station mark (TM 5–441). It is useful in establishing the azimuth of a line and the rectangular coordinates of points at the foot of a vertical shaft. Drill two holes, *A* and *B* at the base of the shaft close to opposite walls (fig. 32). Drive plugs into each drill hole and mark each point with a pencil cross or a tack. Set up the collimator directly over point *A*, using great care in leveling and in centering the instrument. The collimator permits taking a vertical sight upward. Set a tack *C* in this vertical line on a timber fastened across the top of the shaft. Repeat this process with the collimator set over point *B*, setting a point *D* vertically over point *B*. At the surface, jiggle a transit into line with the projected points *C* and *D* and tie in this line by angle and distance measurements with surface control monuments so that the azimuth of *CD* and the rectangular coordinates of these two points can be determined. Since line *AB* is vertically under *CD* it will have the same azimuth as *CD* and points *A* and *B* will have the same coordinates as *C* and *D* respectively. A transit is then set up in the underground passage near the base of the shaft and a traverse line is extended into the passage on true azimuth, using either the jiggling-into-line or the triangulation method (para. 46). The direction theodolite, with prismatic eyepiece attached, may be used in place of a collimator for making the vertical sights upward, using the telescope in both direct and reversed positions. Since the verticality of the line of sight of a collimator depends on the centering of relatively insensitive plate bubbles, this method should not be used in shafts much deeper than 100 feet. The method is more rapid than the use of plumb bobs and wires

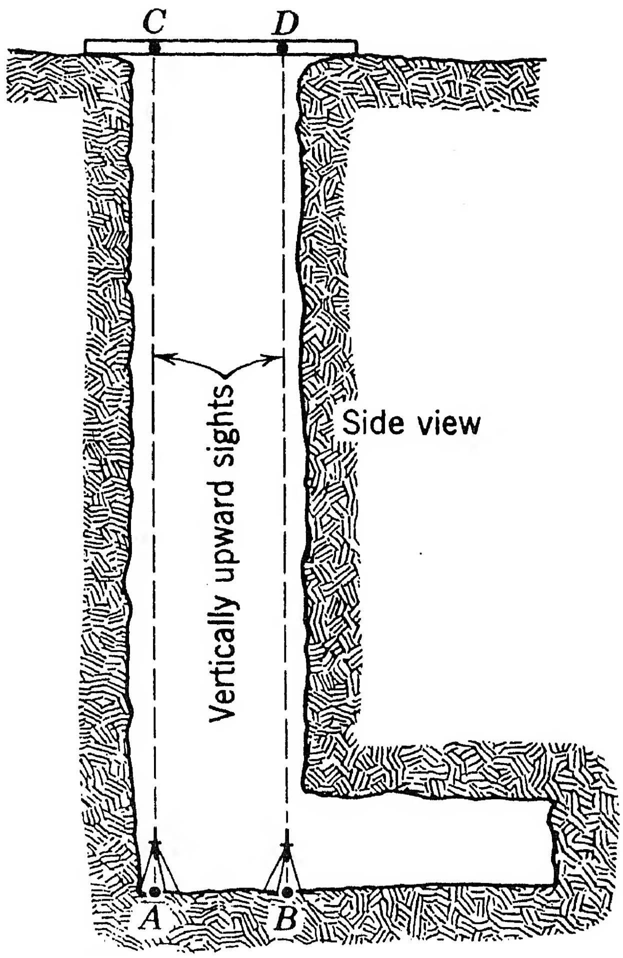


Figure 32. Use of collimator for transfer of azimuth.

and, when carefully performed, can give satisfactory azimuth control for the majority of underground surveys. Plumb bobs and wires are to be preferred for the precise control required in tunnel alinement.

d. Autocollimating Eyepiece. Autocollimation is used whenever extreme accuracy is required.

(1) With a simple autocollimating eyepiece, it is possible to obtain exact alinement of certain optical units with respect to the optical axis of the telescope. Autocollimation is accomplished by projecting the image of the telescope reticle through the telescope objective onto a mirror. The mirror reflects the image back to the telescope and the observer sees a

direct and reflected image of the reticle in the eyepiece. By means of the adjusting screws, the two images can be accurately alined. When the autocollimating eyepiece is installed in a precise theodolite, the reading system of the instrument makes it possible to aline all elements within the theodolite, with as great an angular accuracy as is inherent in the instrument.

(2) Autocollimation has the following advantages:

(*a*) There is no minimum range.

(*b*) The reflecting mirror may be placed directly against the telescope objective.

(*c*) The telescope focusing lens remains set at infinity.

(*d*) Refocusing is not necessary when set at infinity.

(*e*) The chance of errors due to parallax is eliminated.

(*f*) Any deviation from exact alinement represents twice the actual error.

(*g*) Smaller errors can be detected and corrected.

(3) Autocollimation has the following disadvantages:

(*a*) The size of the reflected image decreases with increasing range.

(*b*) It is done more easily at night than in bright sunshine.

(*c*) Requires experience, especially at ranges over (17 m) 50 feet.

(*d*) The quality of the mirror used is quite critical.

(4) To use the autocollimator proceed as follows:

(*a*) Unscrew and remove the telescope eyepiece and replace with the autocollimating eyepiece.

(*b*) Connect the electrical cable of the battery power supply to the autocollimating eyepiece receptacle. The battery power supply can be mounted to a snap stud located on one of the tripod legs.

(*c*) Turn on the lamp power by operating the toggle switch on the battery power supply.

(*d*) Point the telescope at the mirror.

(*e*) Focus the telescope to infinity (rotate focusing sleeve counterclockwise almost to the end of its range).

(*f*) If the telescope is properly pointed, the reflecting image of the reticle will become visible.

(*g*) Eliminate parallax by adjusting the eyepiece focusing ring.

(*h*) Operate the theodolite tangent screws to adjust the alinement of the two reticle images.

Note. Be sure to use the proper replacement incandescent lamp for the autocollimating eyepiece. Both the filament and the glass bulb must be accurately centered. If these requirements are not met, the quality of the reflected image may be poor.

48. Measuring Down Shaft Distances

Direct vertical measurements with a tape and indirect measurements with a plumb-bob wire are described in paragraph 42*a*. Depth measurements in an inclined shaft are measured on the slope between levels, vertical angles being read so that horizontal and vertical distances can be computed.

Section VII. TRAVERSE

49. Selection of Traverse Stations

Although transit traverse points can be set as the work proceeds, it is generally better to carefully select all stations throughout the underground workings before starting the instrumental work. A three-man party is used to make sure that sights are clear between successive points. Stations should be set at the intersection of all passageways in such positions that sights can be taken into all workings. Intermediate stations should be selected to give the longest possible sights. Permanent stations are commonly set in roof plugs (para. 21*c*). Points sometimes must be set in the floor. Where underground traffic is heavy and the roof is low, roof stations should be near a wall to avoid possible disturbance or displacement. If a side telescope must be

used at a station there should be ample clearance between the instrument and the wall.

50. Fixing Stations

Station marks are commonly fixed by driving wood or lead plugs into drill holes in the roof and driving a spad into the plug. For temporary work, spads are driven into roof timbering, spikes are driven into railroad ties, or plugs are inserted in drill holes in the floor.

51. Marking Stations

Stations can be located more readily if they are marked with white or brightly colored paint which contrasts sharply with the color of the rock or ore. The rock around the plug is brushed clean before the paint is applied. It is difficult to keep paint in an open can free of dirt and water. A day's supply carried in a pressure oil can is well protected and not easily spilled. A small quantity can be squirted onto the brush when needed. Distinguishing marks are commonly used, such as a circle for a transit station, a cross for a line point, a square for a bench mark. The station number is either painted on the wall nearby or stamped on a metal disk having a small hole in the center. The spad is driven through this hole into the roof plug.

52. Numbering Stations

The system of numbering used depends on whether the survey is being run in a cave, tunnel, or mine. For a cave survey it may be convenient to number the transit points consecutively. In a tunnel, as on a route survey, it is customary to indicate each transit point by its stationing or distance from the starting point of the survey. In a mine, where there may be several levels, a block of numbers is generally assigned to each level, 100 to 199 for stations in the first level, 200 to 299 for the second level, and so on. Station numbers in cross cuts may be prefixed by the letters XC to distinguish them from main traverse stations. Where the levels extend in opposite directions from a central shaft, the even numbers are often assigned to the stations on one side and the odd numbers to those on the other. Other systems have been used. Where stations are numbered consecutively in the order of their establishment, each number and corresponding station description must be entered in a record book. A good numbering system should permit identification of a station even if its tag has been removed. The system should inform the engineer as to the general location of a station having a given number.

53. Similarity of Surface and Underground Traversing

The same basic methods of traversing used in surface surveys (TM 5–232) are used in running underground traverses. Traverse angles are commonly measured by turning clockwise angles from the backsight to the forward station or by measuring deflection angles. It is customary to double the angle, at least, reversing the telescope between readings. Where lines of sight are steeply inclined and a side telescope is used, four repetitions of the horizontal angle are commonly taken to provide a check (para. 33*c*).

54. Peculiarities of Underground Traversing

Underground traversing differs from transit and tape traversing on the surface in several particulars.

a. Three-Dimensional Coordinates. It is usually necessary to measure the distance on the slope from the transit telescope to the point sighted. This requires observing and recording vertical angles as well as horizontal angles so that the slope distances can be reduced to horizontal lengths and to differences in elevation. The station marks may be either above or below the instrument or the point sighted. It is therefore necessary to measure the height of instrument (hi) and the height of sight (hs) and to record these values with their correct algebraic signs (para. 28) so that the final computation of the traverse will give not only the horizontal coordinates of the stations but their elevations as well. In this respect, underground traversing is analogous to stadia traverse work in surface surveys.

b. Instrumental Setups and Eccentric Telescopes. The limited space in many underground passageways may necessitate low instrument setups or setups very close to a wall. Extension-leg tripods, trivets, brackets, and lateral ad-

justers may be required to meet these conditions (para. 16). Steeply inclined sights necessitate the use of prismatic or auxiliary telescopes for the measurement of both horizontal and vertical angles. Angle measurements with these special telescopes are discussed in paragraphs 31 through 41.

55. Three-Tripod Method of Traversing

This method involves the use of three special tripods having leveling and centering heads similar to the base of a transit. The tripod heads are designed to fit either the transit or two special lamp targets so that the transit or a target can be interchangeably mounted on a tripod. The center of the target is the same distance above the tripod leveling head as is the center of the transit. Each target carries two plate-level vials so that the sight lines of the target can be made truly vertical and horizontal. The tripods are set vertically under or over three adjacent traverse stations, the transit mounted on the central tripod and the targets set at the rear and forward stations. When all angles have been measured at a station, the transit and the forward target are interchanged on their respective tripods and the rear tripod and target is carried forward to the next station. This method has the advantage of freeing two men, usually needed to give line or to illuminate a plumb bob, for other duties while the transit man is measuring and repeating the horizontal and vertical angles. It also provides nearly ideal conditions for the accurate measurement of angles and gives an opportunity for check measurements of the slope distance, hi, and hs. The directional theodolite provides such interchangeability between instrument and target through the use of the tribrach (TM 5–232). Current developments approaching the standardization phase will provide this feature on other standard engineer survey equipment.

Section VIII. TOPOGRAPHY

56. Nature and Purpose of Work

Surveys of underground topography consist of angle or distance measurements which are taken from a transit line to points on the walls, roof, or floor of passageways to determine the physical dimensions of the workings. For certain purposes, the elevations of these points are also required. Such measurements are necessary for the preparation of maps and models of mines and caves, for the determination of the quantities of ore or rock removed, and for computing the dimensions of tunnels and the quantities of concrete lining placed therein. The methods which are used to obtain these data are covered in paragraphs 57 through 62. The method selected will be determined by the required use and character of the data.

57. Right-Angle Offsets

a. Method. This method is particularly adapted for mine mapping and for measuring the volume of excavated material in drifts and narrow rooms and stopes. A tape is stretched on the floor on line under two adjacent transit stations. At intervals, depending upon the irregularity of the passage, right-angle offsets are measured from the transit line to both walls, using a level rod, folding rule, or metallic tape. Offsets are also taken at such other intermediate points as may be necessary to correctly show the true shape of the drift, room or stope. Such points would locate projections of the walls, pillars, chutes, and other important features. For mapping purposes, distances to the nearest foot will generally suffice. For quantity estimates, measurements should be to $\frac{1}{10}$ foot. The horizontal offsets are usually measured at about waist height. In most mine passages the walls will be nearly vertical and errors caused by irregularities in the walls will tend to be compensating. In caverns, where walls are less regular, the maximum and minimum distances to the wall at any height at each offset are measured and plotted by special symbols on the final map. The distances to the roof and floor are measured at each offset point by setting the transit under the first station, sighting the second, and then taking the readings where the line of sight intersects a level rod held first, in inverted position, against the roof and then, in direct position, on the floor. These measurements are often made at the

same time that an underground traverse is being run and may be entered in the traverse notes.

b. Note Entries. The most satisfactory form of field-note entry is to show the actual values of the offset measurements in appropriate positions on the right-hand page of the notebook, opposite the station at which the offsets were measured, as shown below:

		3.1	
10 + 60	5.2		4.6
		4.2	

This indicates clearly that, at station 10 + 60, the offset measured to the left wall was 5.2 feet, that to the roof was 3.1 feet, that to the floor was 4.2 feet, and the offset to the right wall was 4.6 feet. Such notes should read *up the page* starting with the lowest station at the bottom so that values to the right, as one looks along the transit line in the direction of increasing stationing, are shown on the right-hand side of the page of notes.

58. Radial Offsets

In tunnel work, where accurate determination of the volume of rock excavation is often required, radial offsets are measured to provide more detailed information concerning the cross-sectional area and the distances from the transit line to points on the perimeter. A device (fig. 33) is used for determining the angle at which each offset is measured. The graduated disk, mounted vertically on a tripod, is brought into coincidence with the transit line, both horizontally and vertically, at each station where offsets are to be measured. This instrument, radial-offset dial (called a sunflower machine), has a centrally pivoted arm which serves as a guide for a graduated measuring rod of sufficient length to reach from the instrument to any point on the perimeter of the tunnel. Offsets are then measured from the instrument to as many points as necessary to correctly portray the perimeter, and the angles and distances are recorded. Frequently these values are plotted directly to scale on a sheet of polar-coordinate paper, a separate sheet being used at each station. Where there is relatively little of this work to be done, or where a sunflower machine is not available, a radial-offset dial can be improvised by clamping a 12-inch-square (0.3-m-sq.) piece of board to the target of a level rod, tacking a pad of polar-coordinate paper to this, and driving a finishing nail part way into the board to mark the center from which the offsets are taken. One man holds the level rod at a station at which measurements are to be made and raises or lowers the target bearing the wooden square until the instrumentman indicates that the central nail is on the transit line. Another man then measures offsets from this nail to points on the periphery of the tunnel and plots the offsets to scale on the polar-coordinate sheet. Sheets showing the offsets at successive stations must be removed from the pad with care to avoid tearing them on the finishing nail.

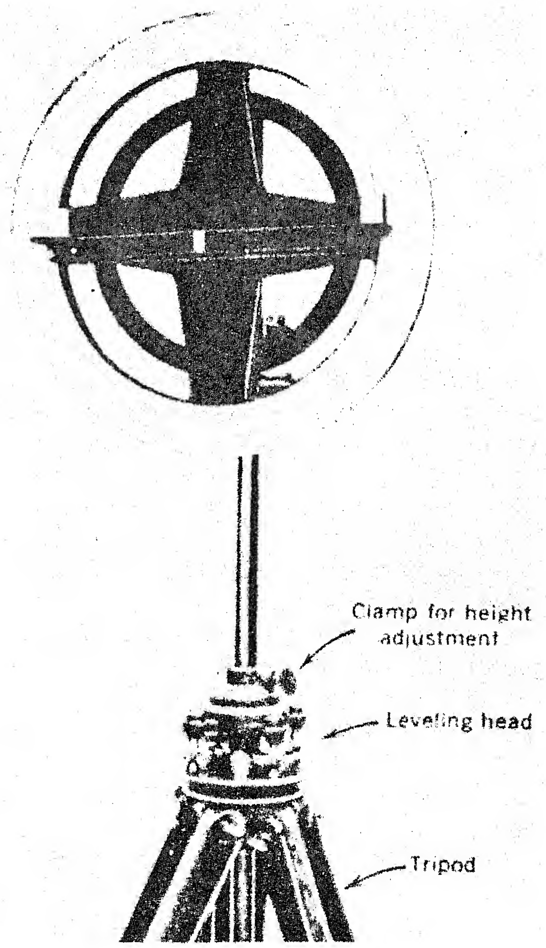


Figure 33. Radial offset dial (sunflower machine).

59. Angles and Distances

Where large, open chambers are encountered in caves and mines, underground topography can be obtained conveniently by measurements of angles and distances from transit stations to points on the periphery. The method is analogous to that of transit and tape or stadia topography on the surface. With the transit set up under a station and correctly oriented in azimuth or reading zero on another station, the azimuth or horizontal angle, the vertical angle, and the distance are measured to each point which is to be located. A light, attached to a long rod if necessary, is commonly held at each point for sighting purposes. Distances may be taped or measured by stadia.

60. Tape Measurements

In broad, low-ceilinged chambers, unobstructed by pillars or roof supports, a point can be located by ties from two transit stations. The point is plotted at the intersection of arcs swung from the plotted positions of the two stations. Where the slope is too great to permit horizontal taping, the measurements are made on the slope and reduced to horizontal by use of clinometer readings of the slope.

61. Angle Measurements

Points for topography in large, open chambers can be located by angles measured from two transit stations. Figure 34 shows a plan and two elevation views of a typical case. The transit line is AB. This distance and the elevations of A and B are known. At A the horizontal angle BAC and the vertical angle (α) to C are measured. The corresponding angles are recorded at B. Two transits can be used, the angles being turned simultaneously to a light held at C, or all angles can be measured with a single transit, occupying A and B in succession. The two horizontal angles suffice to locate the position of C from the plotted positions of A and B. The horizontal distances AC and BC can be computed or scaled from the plotted map. The differences in elevation between A and C and between B and C are then determined by multiplying the respective horizontal distances by the tangents of the vertical angles α and β and applying the height of instrument in each case. Thus, two independent determinations of the elevation of point C are made. The distances involved normally are so shart that no correction for curvature and refraction need be introduced. This procedure for determining the elevation of a point is identical with the trigonometric leveling method widely used in plane-table work and in determining the elevations of triangulation stations (para. 27 and TM's 5–232 and 5–441).

62. Practical Uses of Photography

In reports on tunnel construction, mining operations, or the utilization of caves for military purposes, a clearer concept of the underground topography is obtainable if the report is illustrated with photographs taken in the underground passageways with the aid of flashbulbs. These photographs may be provided with transparent overlays on which are indicated the important features to which it is desired to call attention. The photographs should include people, level rods, or readily recognizable objects for purposes of determining scale, since tunnels or caverns are generally devoid of natural features with recognizable dimensions.

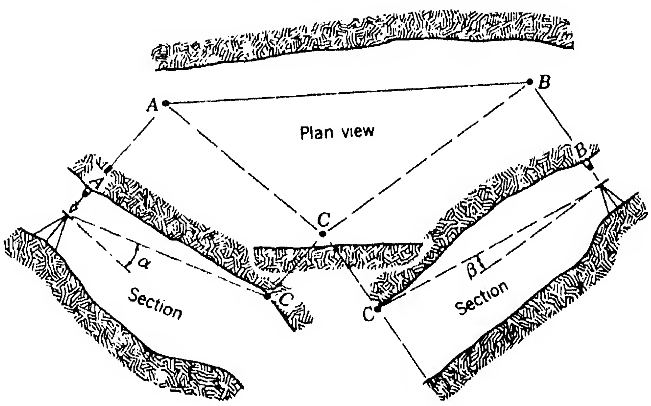


Figure 34. Point location by angles from two stations.

63. Two-Man Surveys

A single individual may run exploratory or reconnaissance surveys in caves and mines, but the danger in working alone in unventilated passages and in using old ladders in abandoned workings seldom warrant it. Lines and grades normally are determined by use of a forestry compass and clinometer or with the pocket transit, the instrument cradled in the hand, or supported on a box, or mounted on a staff. Distances are measured with a metallic tape, with the zero end of the tape anchored by a cord hooked on a nail, rock, or timber. Short measurements are made with a folding or flexible rule. Lamps are used for station targets. See paragraph 11*a* for items of equipment.

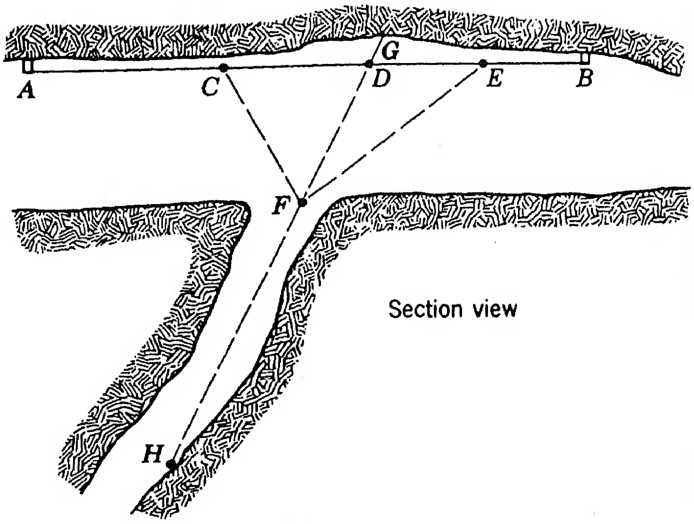


Figure 35. String survey.

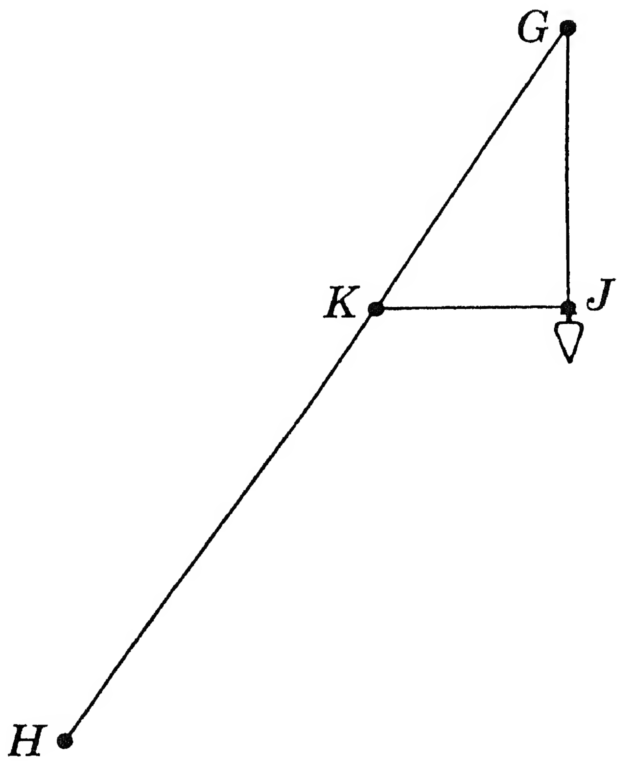


Figure 36. Measuring slope of wire.

64. Pacing Surveys

Military sketching boards are useful for underground reconnaissance surveys. The distances are paced and the sights taken on lamps. Procedures are essentially the same as in surface surveys of this type. In confined quarters, the tripod may be dispensed with and the board held on the left arm by a strap.

65. String Surveys

Auxiliary traverses, to provide control for plotting steeply inclined and tortuous passages, have been extended from a main-control traverse in a drift into such passages by means of string surveys. No angles are measured, a sufficient number of distances being taped to provide data from which the traverse angles can be determined either graphically or analytically.

a. Data Required to Determine Azimuth. Figure 35 shows stations *A* and *B* on a main traverse in a drift. The coordinates and elevations of these points and the azimuth of the line joining them are known. A wire is stretched between *A* and *B* and bits of soft lead or solder are clamped to the wire with pliers at conveniently selected points *C* and *E*. A second wire, *GH*, is extended from *G* into the winze at *H*, being so secured that it just touches the first wire at *D*. Points *D* and *F* are marked in the same manner as *C* and *E*, point *F* being conveniently chosen so that *DF* is as long as possible. Horizontal distances *AC*, *CD*, *CF*, and *DF* are taped. With these data

known, the horizontal projection of the figure can be plotted, or the horizontal angles of the triangle CDF can be computed and the azimuth of DF and the coordinates of D and F can be computed or determined graphically. If the distances DE and FE are also taped, a check is obtained.

b. Data Required to Determine Slope and Elevations. The slope of the wire GH (fig. 35) is determined by hanging a plumb bob from G and measuring the distances GJ, GK, and JK as indicated in figure 36. The slope distance from D to H (fig. 35) is then measured. Since the slope and azimuth of this line are now known, both the coordinates and elevation of point H can be determined graphically or analytically.

c. Additional Data for Vertical Angle in Passage. When the passage turns through a vertical angle (fig. 37), the cord or wire GH is not stretched tight. A plumb bob or weight is hung on the wire as indicated so that the wire clears the roof of the passage. The length and slope of the two segments, GL and LH, are then measured as indicated above, the azimuth of the two segments remaining the same. The coordinates and elevation of H can then be determined by plotting the horizontal and vertical projections of the figure or by computations.

d. Additional Data for Horizontal Angle in Passage. Where the passage has a turn in its horizontal alinement (fig. 38), two cords or wires, CE and FJ, are stretched in the passage in such a manner that they just touch at G. Horizontal distances are measured from G to H, H to E, and E to G. These suffice to determine the horizontal angle at G. Slope measurements are made from D to G and from G to J and the slopes of these lines are determined as indicated in *b* above. These measurements supply all data required to determine the azimuth of line GJ and the coordinates and elevation of J.

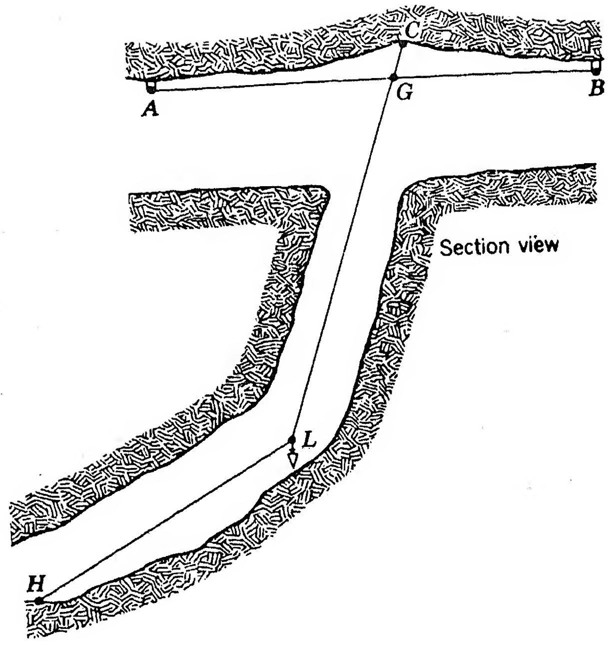


Figure 37. String survey in passage of changing slope.

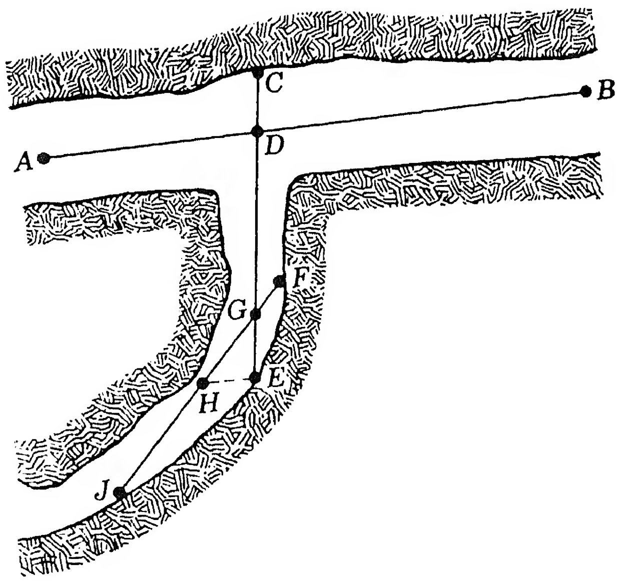


Figure 38. Plan of string survey.

66. Traversing Steep Workings Without Auxiliary Telescope

Figure 39 shows a steeply inclined shaft connecting workings at two levels. A traverse has been run in the upper drift, and the true bearing of the line AB and the horizontal and vertical coordinates of these stations have been determined. It is required to establish the azimuth of a line and the coordinates of a station in the lower level. The transit is set up under station A and sighted on station B. A wire is stretched as tight as possible between any two convenient points such as C and D.

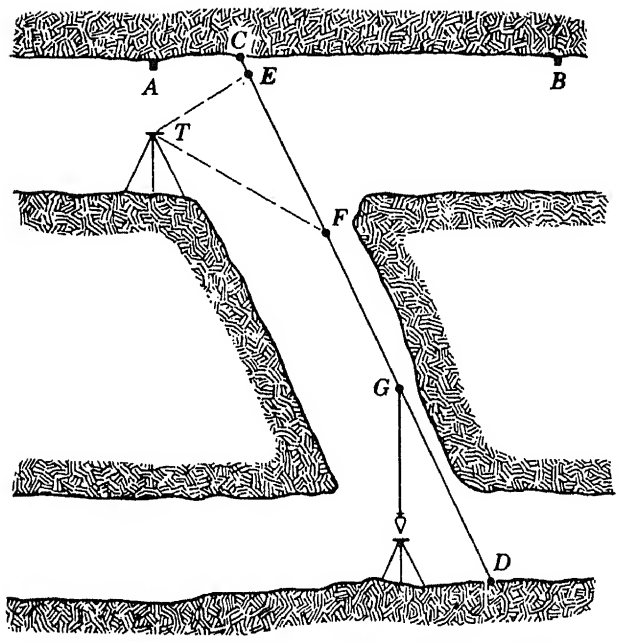


Figure 39. Traversing shaft without auxiliary telescope.

Small bits of lead or solder are clamped to the wire at E, F, and G, using a pair of pliers. Horizontal and vertical angles are measured from the transit to points E and F. The slope distances TE, TF, FG, and GD are also taped. From the first two distances and the horizontal and vertical angles, the coordinates of points E and F can be computed. From these coordinates, the slope and azimuth of the line CD are computed. This slope and azimuth, together with the distances FG and GD, will furnish the coordinates of G and D. A transit can then be set up under a plumb bob hung from G, sighted on D, and the traverse continued on the lower level from a station of known coordinates and from a line of known bearing. Because of inevitable sag in the wire, the method should not be used when the distance CD is large. On very steep slopes, CD may be as great as 150 feet (50 m) without causing appreciable error. As CD becomes flatter, this distance should not exceed 100 feet (30 m).

67. Projecting Azimuth Down Inclined Shaft Using Prismatic Eyepiece on Main Telescope

Figure 40 shows a steeply inclined shaft leading from the surface to a lower level. The transit available for the survey has a prismatic eyepiece and hence is capable of sighting steeply upward but not downward. The transit is first set over point A, a control station on the surface, and sighted at B, the azimuth of the line AB being known. This line is transferred to points C and D on the shaft collar. A wire is stretched tightly between C and D and a heavy plumb bob is hung from C, its motion being dampened by a container of oil secured to the footwall of the shaft. The transit is then set up at the foot of the shaft and jiggled into line until the vertical crosshair will travel along the full length of the wire CD and the plumb-bob cord CE. It may prove more convenient to stretch two wires, about one-quarter inch apart, on either side of points C and D, and shift the transit until the vertical crosshair will follow the bob cord CE throughout its length and bisect the space between the two wires joining C and D.

68. The Stadia Method

The stadia method has definite application to underground surveys, not only for locating underground topography, but for traverses when time is limited and when stadia accuracy will meet the requirements of the survey. The short level-rods employed for underground spirit leveling are satisfactory for use as stadia rods since sights normally are short and the graduations of these rods are not difficult to read at short distances. Where the illumination is poor, it may prove necessary to place two targets on the rods so that the intercept can be determined with accuracy. Since many of the vertical angles measured are large, it is particularly essential that the rod be held plumb to avoid serious errors in computations of distances and elevations. The rodman should be provided with a level to aid him in keeping the rod vertical. Measurements of horizontal and vertical angles and rod intercepts are made in the same manner as in surface stadia surveys. Since sights are taken to both roof and floor stations and the rod may be held in both the normal and inverted positions, it is well to draw a sketch similar to figure 41 to aid in computing the elevation of a station. In this case the transit is set 4.2 feet below station A

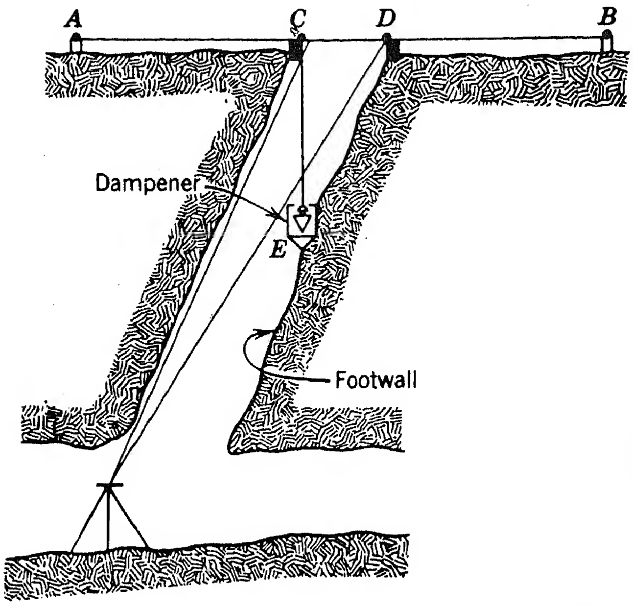


Figure 40. Transferring azimuth down inclined shaft.

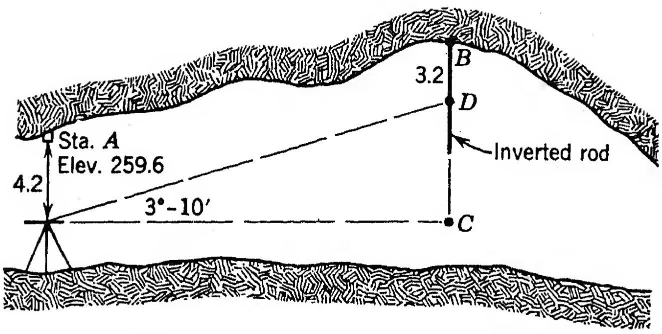


Figure 41. Elevation and distance by stadia.

which has an elevation of 259.6 feet. With the rod held inverted on point B, an intercept of 1.88 is read and a vertical angle of 3° 10′ is read with the middle crosshair on 3.2 feet. The stadia constant of the instrument is 100 and the $F + c$ factor is negligible for the interior-focusing transit used. Using stadia tables (TM 5–236), the horizontal distance from A to B is found to be 188 feet and the difference in elevation, CD, is 10.4 feet. The elevation of B is then equal to 259.6 — 4.2 + 10.4 + 3.2 = 269.0 feet.

Section X. PROBLEMS, NOTES, AND COMPUTATIONS

69. Alinement of Curves

Surveys for alining curves in tunnels and other underground workings are based upon the principles and methods used in route surveying (TM 5–233).

70. Transferring Azimuth by Wire and Plumb Bobs

The stopes or raises extending above or below a mine level or between levels are frequently crooked so that it may be impossible to make direct instrumental sights through them. Figure 42, which shows a steeply inclined stope, indicates a method which may be used to transfer azimuth in such a case. The azimuth of the line BA is known. A transit is set up under station B and oriented on A. A wire or cord is hung from B to a point C in the stope and kept taut by one or more plumb bobs or weights as indicated. The point of support at C is shifted laterally until all of the wire which can be viewed through the telescope is in line with the vertical crosshair. The wire BC then has the same azimuth as the line BA. While the transit is still at B, point D is set in line and the vertical angle and slope distance are measured from the instrument to D. These values are used to obtain the coordinates of D from the known coordinates of B. The transit is now set up under C, backsighted on D, checked by alining on the wire, and the survey is continued in the stope from a line of known azimuth and from a point of known coordinates. If the passage is not reasonably parallel to the main level, it may not be possible to establish the wire BC in the same vertical plane with the line AB. In this case, it will be necessary to measure both horizontal and vertical angles at B to establish the azimuth of the traverse line and the coordinates of the points.

71. Giving Line

Survey parties must not only give line to the instrumentman during survey operations but must also furnish points from which mine or construction crews may extend their operations. This involves establishing two or more points from which plumb bobs can be hung

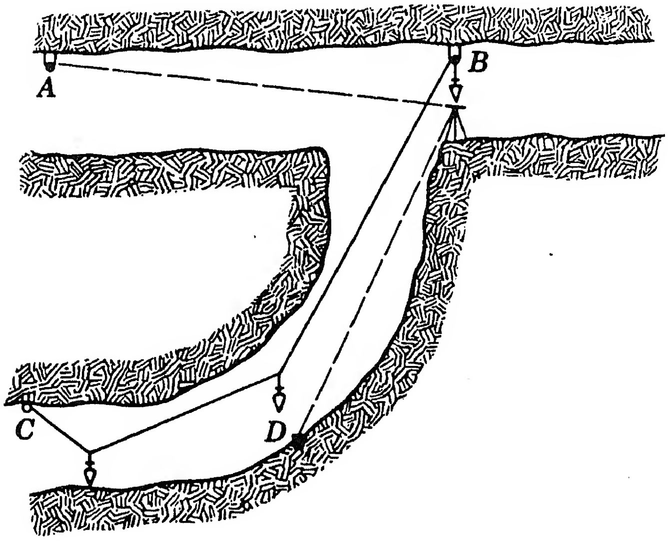


Figure 42. Transferring azimuth by wire and plumb bob.

to define the line and grade on which excavation is to proceed. With the transit set up and oriented on the correct azimuth, spads are set in the roof some 10 to 20 feet apart along the line ahead. Plumb bobs are then hung from these to serve as sights to control the direction of excavation of the heading. If the passage is on a grade, the plumb bob cords must be of a definite length so that a line sighted along the tops of the plumb bobs will define the grade.

72. Establishing Line and Grade for a Crosscut or Connection

In a mine, it is frequently necessary to drive connections from one drift to another or to excavate crosscuts from a gangway or drift approximately at right angles to the strike of a vein. In tunnel construction, connections are also required when a pioneer tunnel is driven parallel to the main tunnel. The points from which connections or crosscuts are to be driven must be fixed by the survey party and the line and grade of the proposed passage must be determined. Figure 43 shows two drifts running roughly parallel to the strike of a vein of ore. It is desired to drive a connection between *A* and *E* for mine development. The coordinates of *A* and the bearing of the line *AB* are known. The azimuth and grade of the line *AE* are required so that excavation of the connection can proceed from both drifts. A traverse *ABCDE* is extended through the connecting passages of the mine between points *A* and *E* which have been selected as the two ends of the connection. The length and bearing of the closing side *EA* of this traverse are computed. The grade of the connection is determined from the length of this line and the elevations of the drifts at *A* and *E*. The methods of computation of such a line are outlined in paragraph 68. The survey party then drives two plugs and spads in the roofs of the drifts on line spaced about *A* and *E* and hangs plumb bobs of known cord length from them to define the direction and grade of the connection (para. 70). Since the connection is nearly perpendicular to the drifts, the plumb lines in each drift will necessarily be quite close together. Additional points on line and grade should be instrumentally established in the connection as excavation proceeds. It is recommended that when setting grades for a crosscut or haulage way, the grade be set just before the grading crew starts the excavation. Two methods for marking these grade points are—

a. Driving a nail or spike into the wall and painting around the nail.

b. Painting a small point on the wall.

73. Extending a Traverse Past a Point Inaccessible for Transit Setup

In running an underground traverse through levels and winzes or raises, the situation occasionally develops where a permanent traverse station must be established at a point under which the transit cannot be set up and yet past which it is necessary to carry the traverse. Figure 44 illustrates such a situation. The traverse is to be carried from *A* to *B* to *C* to *D*. The transit cannot be set up under point *B*. A temporary setup can be made at *E*, pending completion to full width of the

rock excavation in the raise between C and E. Point E is temporarily established from the previous station at A, the azimuth and distance from A being measured. If the distance can be measured horizontally from A there is no need to measure the vertical angle from A to E since the elevation of C can be obtained from the elevation of B. The instrument is then set over the temporary station at E, backsighted on A and then sighted on C, preferably through the top telescope. Both the horizontal and vertical angles to C are recorded. The vertical angle must be reduced to center (para. 39). The telescope is then raised and point B is established in the same vertical plane as E and C. A plumb bob is hung from B with its tip F at the elevation of the transit telescope at E. The horizontal distance from B to the transit at E and the slope distance from B to C are then measured. Since BG equals FE plus hi/tan $_a$ and BC and $_a$ have been measured, the vertical angle at B can be computed. With the transit set up back at A, the horizontal and vertical angles and slope distance to B are measured. The coordinates of B are computed and the coordinates of C are then determined, using the slope distance BC and the vertical angle at B. The transit is then set up at C, backsighted on line CE, the azimuth of which is now known, and the traverse is continued to D.

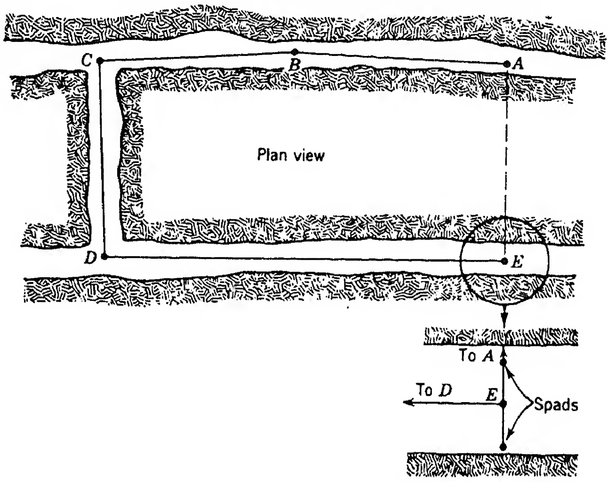


Figure 43. Establishing line and grade for a connection.

74. Field Notebooks

It is the practice in the Corps of Engineers to keep all survey notes in bound books. Many

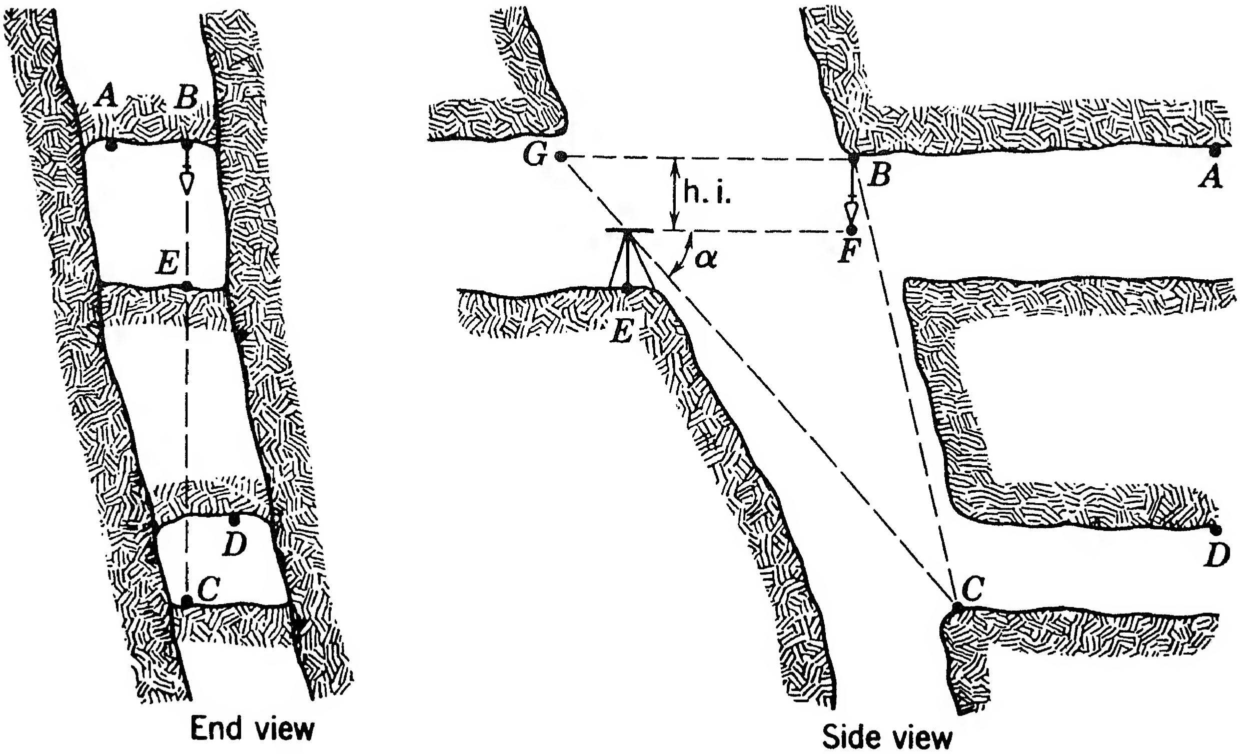


Figure 44. Carrying a traverse past a point inaccessible for transit setup.

valuable survey notes may be rendered illegible from exposure to dirt and dampness if a bound notebook is carried underground day after day. When bound books are used under such conditions they should be of relatively few pages so that a single notebook is exposed to adverse conditions for only a short time. Every effort must be made to protect the notes from damage. A good practice is to keep duplicate notes. One book is used underground to record the information as it is obtained; however, due to the adverse conditions to which it is subjected a duplicate is kept above ground. Daily transfer of the notes to the duplicate set will insure a legible set of notes.

75. Forms of Notes

Survey notes must give all necessary information in such form as to be clearly understandable by both the field party and the office force. Office personnel engaged in survey computations may not see the actual conditions in the field. It is particularly important that all possibility of misunderstanding or misinterpretation of notes by such personnel be eliminated.

76. Notes for Deflection-Angle Traverse (Horizontal Taping)

Figure 45 shows a portion of a page of sample notes for a deflection angle traverse in

Traverse Notes ~ First Level ~ Henry Hill Mine ~ Ringwood, N.J.											
Transit	Station	Horiz.	Defl.	Double	½ Double		Remarks:		Date:	9 Sept.	19
Station	Observed	Distance	Angle	Defl. ∡	Defl. ∡				Party:	Cpl. L.	T. Van
						Plane coordinates				Pfc. O.	C. Roth
	100	187.46				Sta.100 =	3478.21 N.,			Pfc. J.	P. Ward
101			05°10'R	10°21'	05°10'30"R		1296.78 E.				
	102	145.21				True bearing Sta. 100					
	101	145.22				to Sta.	101 =				
102			02°04'R	04°08'	02°04'00"R	N 18° 52'30"E					
	103	201.76									
	102	201.78									
103			00°51'L	01°41'	00°50'30"L						
	104	174.32									

Figure 45. Sample notes for deflection angle traverse (Horizontal taping.)

a nearly horizontal passage where the distances between stations are taped horizontally. In this case, the elevations of the stations are to be determined by spirit leveling. The instrument is set at station 101, backsighted on station 100, plunged, and the first deflection angle turned to station 102. The deflection angle is doubled and one-half the doubled angle is used as the value for traverse adjustment. Space is provided for entering the measured distance forward from station 101 to station 102 and for the check distance measured back. If the traverse is eventually closed, as it should be in important work, the last transit setup will be at station 100 and a measurement will be entered for the distance from station 100 to station 101 which should check the first recorded distance of 187.46 feet. If the traverse is not to be closed, this distance should be measured twice and the second value entered in the notes directly under the first. Thus, the notes show checks on all angle and distance measurements. Angle checks are always desirable. A check on distance measurements is particularly desirable when the tape is too short to measure the entire length of a traverse side in a single operation or where the floor of the passage slopes so steeply that the distance must be measured by "breaking chain." Unless check measurements are made, serious errors or blunders may be undetected.

Traverse Notes ~ Second Level ~ Elk Butte Mine ~ Montana

Transit Station	Station Observed	Horiz. Angle	Slope Distance	Vertical Angle	h.s.	h.i.	Remarks:	Date 18 Oct. 19__
								Party: J.B. Burke
							Coordinates Sta. 201	L.S. Long
201	200	00°00'				−4.16	= 4876.34 N.,	H.E. Walter
	202	182 21	118.87	+00°50'	+2.62		1592.86 E	Spad in roof
	200	" "					Elev. Sta. 201 =	
	202	04 42		+00 50	+2.62		5231.46 ft.	
							Bearing Sta. 200	
202	201	00 00				−4.02	to 201 =	
	203	176 05	153.67	+00 36	+2.79		S 48° 10' 30" W	Spad in roof
	201	" "					Horizontal Angles	
	203	352 10		+00 36	+2.79		measured clockwise	
							Transit - Berger	
203	202	00 00				−4.08	#86752	
	204	180 15	169.40	−00 12	+2.40		200 ft. steel tape.	Spad in roof
	202	" "					B.S. 6971	
	204	00 30		−00 12	+2.40			

Figure 46. Sample notes for angle traverse (slope taping).

77. Notes for Angle Traverse (Slope Taping)

Figure 46 shows a portion of a page of sample notes for an angle traverse. Distances between stations are measured on the slope. Horizontal angles are turned clockwise. The notes indicate that the instrument was first set up at station 201 which had coordinates of 4876.34 N., 1592.86 E.; and an elevation of 5231.46 feet. The telescope was 4.16 feet below station 201. It was backsighted on station 200 (true bearing N. 48° 10′ 30″ E.) and a clockwise angle of 182° 21′ was turned to station 202. The telescope was then plunged and, with the initial reading still on the circle, the instrument was again sighted on station 200 and the angle to station 202 was doubled. The point sighted was 2.62 feet below station 202. Vertical angles were read in both the direct and reversed positions. All angle measurements were checked. It might appear that there was no check on the single recording of the slope distance of 118.87 feet. Where the entire side of a traverse is measured in one tape-length, there is ample opportunity to check the measurement. Tension is applied to the tape and reading made. The pull is then slackened off, taken up again, and a second measurement made, the value being entered in the notes when the chainmen are satisfied of the correct reading. When a side telescope must be used for angle measurements, four repetitions of the horizontal angle are required to provide a check (para. 33). The notes may be entered in the form shown, using additional lines at each station, or the typical form for triangulation notes may be used (TM 5–441), with additional columns for slope distances, vertical angles, hs, and hi.

78. Notes for Underground Spirit Leveling

See paragraph 24 and figure 18.

79. Notes for Levels From Slope Distances

Figure 46 shows sample notes for a typical traverse from which the elevations of the stations are determined from the slope distances and vertical angles. Figure 47 shows a sketch of the readings taken from station 201 to station 202 as indicated by these notes. The difference in elevation between the telescope and the point sighted is 118.87 sin 0° 50′ = 1.73 feet. The elevation of station 202 is therefore 5231.46 — 4.16 + 1.73 + 2.62 = 5231.65 feet. The horizontal distance from station 201 to station 202 is 118.87 cos 0° 50′ = 118.86 feet. This distance may also be computed by applying the horizontal correction to the slope distance, this correction being 118.87 vers 0° 50′.

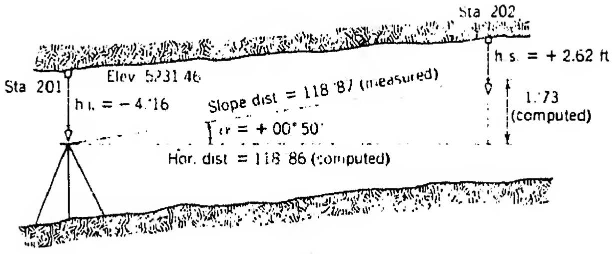


Figure 47. Elevations from slope distances.

80. Supplementary Sketches

Underground surveys require a variety of sketches to supplement the notes and to present data not shown. A double page of the notebook is required (fig. 46) to show the notes for several successive stations of an underground traverse. This double page is followed by another double page (fig. 48) on which one or more sketches are drawn to show a rough outline of the underground passage in relation to the transit lines. Where a traverse is run through a cave or abandoned mine workings, similar sketches will show, to approximate scale, the extent of openings in the cave or the outline of entries and rooms in the mine which may be utilized for storage, shelter, or other military purposes.

81. Computations for Underground Work

The following paragraphs describe the procedures involved in computing underground traverses, the computational work connected with a number of underground surveying problems, and the determination of the volumes of material excavated from underground passages.

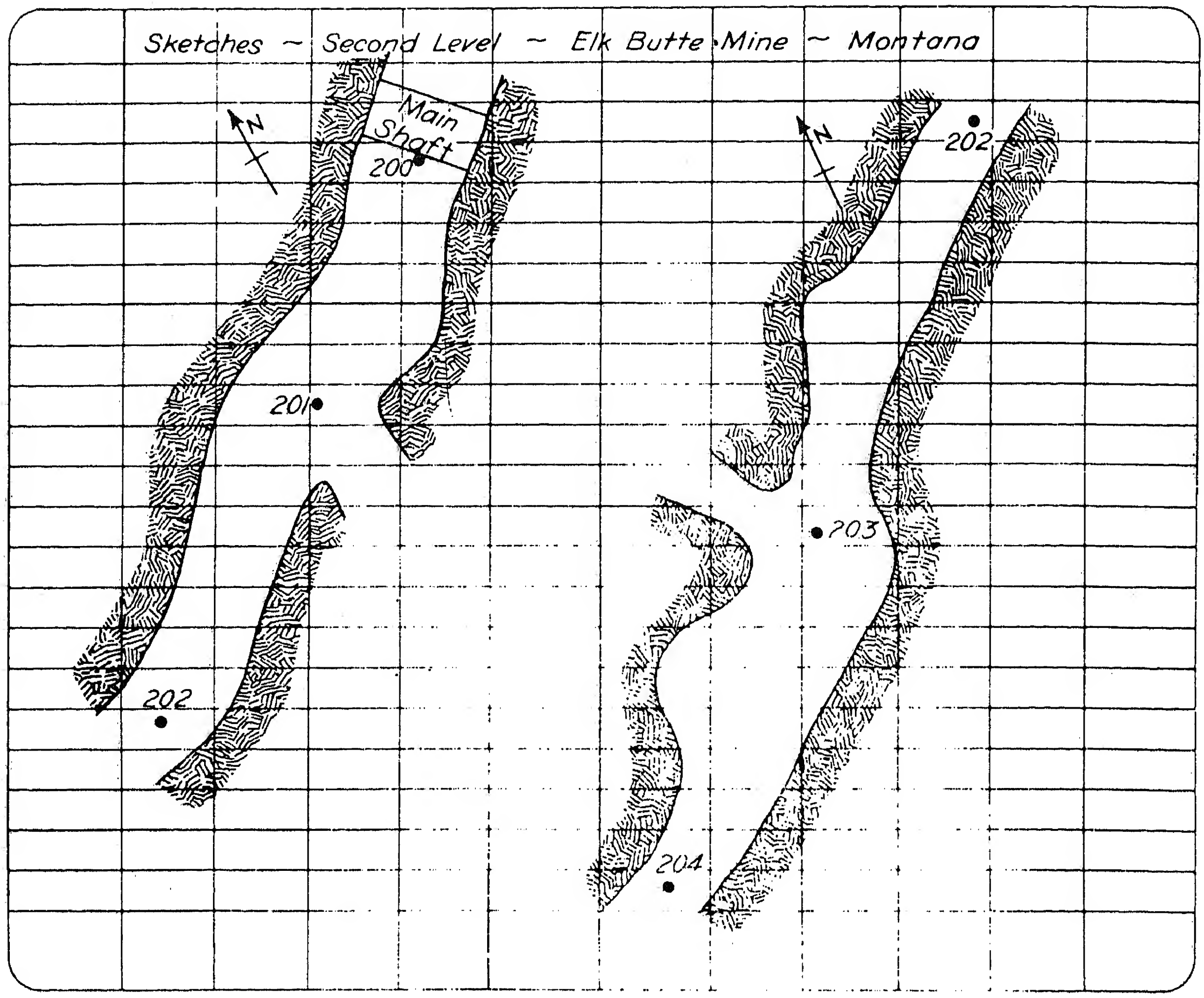


Figure 48. Supplementary sketches.

82. Rectangular-Grid Systems for Coordinate Computation

a. A rectangular grid is composed of two sets of uniformly spaced straight lines intersecting at right angles. These lines form a plane rectangular-coordinate system for use in traverse and triangulation computations and to assist in the accurate plotting of the survey stations on a map. The computed rectangular coordinates of the survey stations show the relative position of each station with respect to all other stations computed on the same grid system.

b. A rectangular-grid system may be a local system based upon designated coordinates for one survey station, or it may be a universal system based upon geographical subdivisions of the earth as described in TM 5-241. For plane surveys, a local grid sysem is often used; however, where ties to geodetic control can be made, it is preferable to use the Universal Transverse Mercator Grid (TM 5-241-1), so that the position of each survey station will be known with respect to survey stations throughout the world.

c. A local plane-grid system designed to cover an area normally would not extend more than 4 or 5 (about 7 km) miles from the center of the area. In the case of a long traverse extending in a general direction such as west, and if the grid band is not over about 5 miles (8 km) in width, the extension can be of practically any length.

d. Many underground surveys are based on a local grid system referred to coordinate axes passing through a control monument located at the entrance to an adit, at a tunnel portal, or at the top of a shaft. These coordinate axes run north and south, and east and west, preferably with reference to the true meridian passing through the control station. This station is arbitrarily assigned coordinates of such magnitude that the coordinates of all stations in the system will be positive. The coordinates of survey stations referred to in this section, and elsewhere in this manual, are based upon such a grid system.

83. To Establish the Line Connecting Two Points

The two points are designated A and B. Assume that the horizontal and vertical coordinates of A are known. Run a traverse through connecting underground passages from A to B. To provide a check, close the traverse back to A, using different stations. From the traverse data, compute the horizontal and vertical coordinates of B. Let x_a, y_a and z_a represent the total departure, total latitude, and elevation of A, and let x_b, y_b, and z_b represent the corresponding values for B. The following quantities can then be determined.

a. Bearing of Line AB. The bearing angle, β, of the line AB is given by the expression

$$\tan \beta = \frac{x_a - x_b}{y_a - y_b}$$

The quadrant for the bearing is determined by inspection or by application of the following rules. When both the numerator and denominator in the above equation are positive, the bearing is in the northeast quadrant. When the numerator is negative and the denominator is positive, the bearing is northwest. When both numerator and denominator are negative, the bearing is southwest. When the numerator is positive and the denominator is negative, the bearing is southeast.

b. Horizontal Distance AB. The horizontal distance from A to B equals

$$\sqrt{(x_a - x_b)^2 + (y_a - y_b)^2}$$

c. Difference in Elevation AB. The difference in elevation between A and B is equal to $z_a - z_b$.

d. Slope Distance AB. The distance from A to B, measured on the slope, is equal to

$$\sqrt{(x_a - x_b)^2 + (y_a - y_b)^2 + (z_a - z_b)^2}$$

e. Slope of Line AB. The tangent of the angle θ which the line AB makes with the horizontal is given by the expression:

$$\tan \theta = \frac{z_a - z_b}{\sqrt{(x_a - x_b)^2 - (y_a - y_b)^2}}$$

84. Computation of First and Corrected Underground Traverses

In paragraph 45 a description is given of the general problem of transferring a meridian into underground workings by plumbing down two vertical shafts. The underground surveys involve a traverse run on an assumed bearing between the two shafts and the computation of the length and bearing of the closing side. The bearings of this traverse are then adjusted to conform to the known bearing of the closing side as determined by surface surveys, and the traverse is recomputed to furnish the coordinates of the underground stations. Typical computations for such a traverse are given below.

a. First Underground Traverse. Figure 49 shows the plan of an underground traverse connecting points plumbed down two shafts at 1 and 6. Measured distances and deflection angles are indicated on the sketch, as is the assumed bearing of the first course 1–2. The traverse has actually been closed from 6 back to 1, using different stations on the return run, so that a check is available on the coordinates of 6 with respect to 1. In the interest of clarity, only the forward traverse and its computations are shown. The traverse distances are too long for slope taping in this case. The drift is nearly level and all distances have been measured horizontally. The elevations of the traverse stations, which are not shown, are to be determined by spirit leveling. Figure 50 shows the computations for this traverse. The summations of the latitudes and departures indicate that the latitude of the closing side 6–1 must be 116.46 feet south and that the departure of this side must be 876.86 feet west. Application of the approximate equations of paragraph 83 gives the length of the line 6–1 as 884.56 feet and the bearing of this

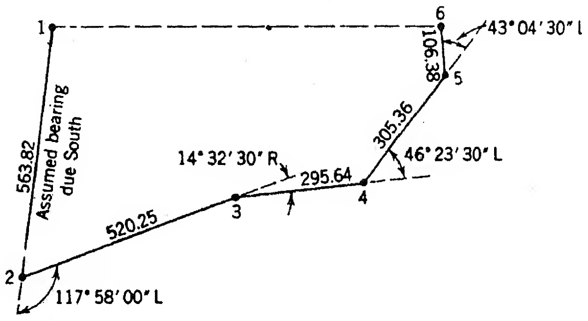


Figure 49. Plan of first underground traverse.

line, with respect to the assumed bearing of 1–2 as S 82° 26′ 00″ W.

b. Corrected Traverse. The computation of the surface traverse run between points vertically above 6 and 1 gives the coordinates of point 1 as 4875.26 N, 3248. 37 E, and the distance 6–1 as 884.58 feet. The latter provides an extremely close check on the distance determined from the underground traverse so that no adjustment of the lengths of the underground traverse (*c* below) is necessary in this instance. The true bearing of line 6–1 is determined by astronomical observations on the surface to be S 75° 12′ 00″ W. Since the assumed bearing of S 82° 26′ 00″ W has been used for this line in the first computation of the underground traverse, a correction angle of 07° 14′ 00″ must be applied to all bearings of the underground traverse. Figure 51 shows the corrected bearings, the final computation of

Computation of First Underground Traverse ~ Shafts 1-6

Line	Deflect. Angle	Bearing	Dist.(ft)	N	S	E	W
1-2		(Assumed) Due South	563.82	—	563.82	—	—
2-3	117°58′00″L	N 62°02′00″E	520.25	243.98	—	459.50	
3-4	14°32′30″R	N 76°34′30″E	295.64	68.64	—	287.56	
4-5	46°23′30″L	N 30°11′00″E	305.36	263.96	—	153.53	
5-6	43°04′30″L	N 12°53′30″W	106.38	103.70	—	—	23.73
			Sums	680.28	563.82	900.59	23.73
				563.82		23.73	
			Diff.	116.46		876.86	

tan. Bearing 6-1 = 876.86/116.46 = 7.52928

Bearing 6-1 = S 82° 26′ 00″ W (to nearest 30″)

Distance 6-1 = $\sqrt{876.86^2 + 116.46^2}$ = 884.56 feet.

Figure 50. Computations for first underground traverse.

Station	True Bear.	Dist. (Feet)	N	S	E	W	Coordinates N	Coordinates E
1	S 07° 14' 00" E	563.82	——	559.32	70.99	——	4875.26	3248.37
2	N 54° 48' 00" E	520.25	299.90	——	425.14	——	4315.94	3319.36
3	N 69° 20' 30" E	295.64	104.30	——	276.63	——	4615.84	3744.50
4	N 22° 57' 00" E	305.36	281.20	——	119.07	——	4720.14	4021.13
5	N 20° 07' 30" W	106.38	99.88	——	——	36.60	5001.34	4140.20
6	S 75° 12' 00" W	884.56	——	225.96	——	855.23	5101.22	4103.60
			785.28	785.28	891.83	891.83		

Computation of Corrected Underground Traverse ~ Shafts 1-6

Figure 51. Computations for corrected underground traverse.

the underground traverse, and the coordinates of all stations.

c. Adjustment of Traverse Lengths. Such traverses should close to within one part in 5,000. When the traverse closures are within this limit but the length of the common side (6–1, fig. 49) shows appreciable differences in the computations of the surface and underground traverses, adjustment is made as follows. It is assumed that survey conditions are more favorable on the surface. The length and bearing of the common side, as determined from the balanced surface survey, is fixed. The latitudes and departures of the courses of the underground traverse are then balanced without changing the latitude and departure of this common side. From these balanced latitudes and departures, the adjusted lengths and bearings of the sides of the underground traverse can be computed (TM 5–232).

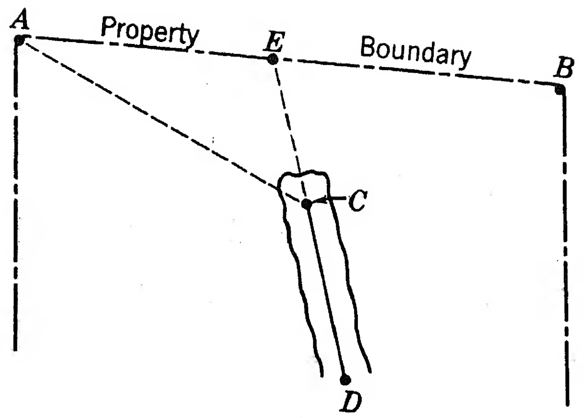


Figure 52. Determination of point where a drift will intersect a boundary line.

85. To Determine the Point at Which a Given Line Intersects Another Line

Figure 52 shows a property boundary AB and a traverse line DC in a drift. The bearings of AB and DC and the coordinates of all four points are known. It is required to determine the distance from C to point E at which the drift, when extended, will meet the boundary line. From the difference, in coordinates of A and C, compute the length and bearing of line CA, using the appropriate equations from paragraph 83. All angles of the triangle AEC are then determined from the differences in the bearings of the intersecting sides. The distance CE is then equal to

$$\frac{AC \sin A}{\sin E}$$

86. Locating Corners of a Shaft to Connect With a Winze

Figure 53 shows drifts at two levels leading from the main shaft of a mine. A winze has already been sunk from the first to the second level. It is desired to sink a second shaft from the surface to connect with this winze. This proposed shaft is indicated by the dotted lines in the figure. The shaft corners must be located on the surface so that shaft-sinking operations can be started. Points A and B represent surface control points. C and D are control stations in the first level. The coordinates of these points and the true bearings of the lines AB and CD are known. A transit is set up under station D, oriented on C, and angles and distances are measured to the four corners of the winze at the first level. The coordinates of these corners are then computed. A surface traverse, $ABEF$ is then run to point F which is estimated to be near the site of the proposed shaft. The coordinates of F are computed. The differences between the horizontal coordinates of F and the horizontal coordinates of four points vertically above the corners of the winze permit the computation of the bearings and distances from F to each of the shaft corners. These corners can then be staked out on the surface.

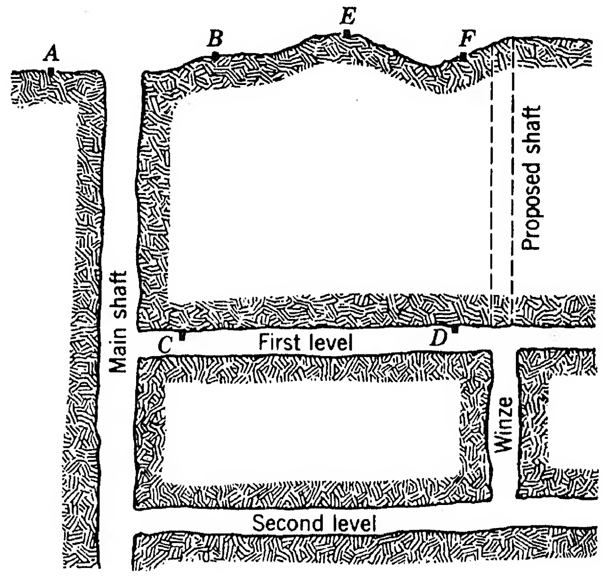


Figure 53. Location of shaft corners.

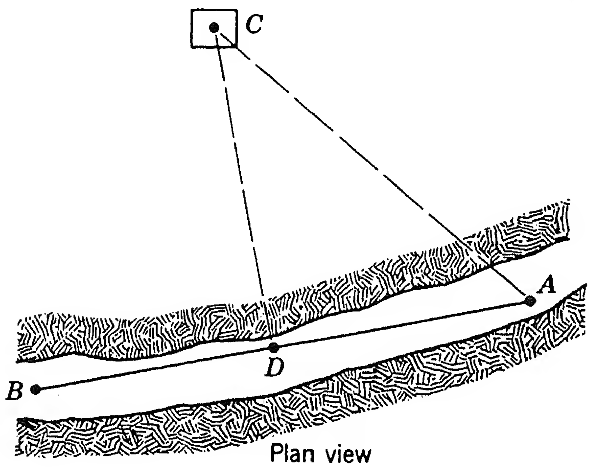


Figure 54. Shortest crosscut from a drift to a shaft.

87. To Determine the Point on a Given Line From Which to Make the Shortest Connection to a Given Point

Figure 54 shows a traverse line AB in a drift. A shaft has been sunk at C. The coordinates of A, B, and C and the bearing of AB are known. It is required to locate point D from which the shortest crosscut can be driven from the drift to the shaft and to determine the length and bearing of this line. For minimum distance, DC must be perpendicular to AB, hence the bearing of DC is readily computed. From the differences in coordinates of C and A, compute the length and bearing of line AC. The bearings of all three sides of the right triangle ADC are then known and the angles at A and C can be computed. The length of AD equals $AC \cos A$ so that point D can be set on line AB. The distance DC equals $AC \sin A$.

88. Computation of Volume

The volume of material removed during the excavation of underground passages is computed from data obtained in underground topographic surveys. As indicated in paragraphs 56 through 62, offsets or other measurements are taken to determine the cross-sectional dimensions of the workings at short intervals. The area of each section is either computed or the measurements are plotted to scale and the area determined by planimeter. The volume between sections is commonly computed by the end-area formula (TM 5–233).

Section XI. MAPS AND MODELS OF UNDERGROUND PASSAGES

89. Maps

The utilization of underground passageways for military purposes requires the preparation of a variety of maps and sectional and perspective views. These include surface plans, plans of the workings at various levels, overlays, vertical cross-sections, longitudinal projections, perspective drawings, and plans or other views to show special features and equipment. The original drawings, particularly where several colors are to be used, are commonly prepared on the best grade of mounted drawing paper. Tracings are made on cloth or paper so that prints can be reproduced as required. To properly show the requisite details, such maps and drawings should be plotted on a relatively large scale. Where the area extent of the workings is limited, a single sheet or roll of drawing paper can be used for the surface map and for each plan of an underground level. In many instances, the workings cover an area of many square miles. Under these conditions, it is desirable to prepare a small-scale key map showing the extent of the passages, and to divide the total area covered into a number of quadrangles. The size of the quadrangles should be such that they can be plotted to the desired scale on sheets of convenient size for filing and for field and office use. Each quadrangle is identified by coordinates or by a reference number on the key map. Several maps may be drawn for a single quadrangle, one showing surface conditions and each of the others representing a different underground level.

90. Surface Plan

The surface plan should show topography, structures, physical features, property lines, improvements, shaft openings, adits, and portals.

91. Level Sheets

Each map of a quadrangle showing the workings at a particular level is plotted by coordinates so that it will register with the corresponding surface sheet and with the maps of levels above or below it. These maps show all openings and passageways at the given level. When plotted on tracing cloth, the map of one level may be overlaid on the map of the level or levels beneath it to aid in the visualization of the relative locations of the various drifts and other workings. Additional maps of intermediate levels or stopes are prepared when necessary to correctly portray all underground openings.

92. Vertical Cross-Sections

Vertical cross-sectional views through the passages normally are taken on planes per-

pendicular to the general direction of the passages. All openings cut by the section are shown on such a view.

93. Longitudinal Projections

These are drawn as projections of workings on a plane generally parallel to the direction of the main passageways. The projection may be on a vertical plane or, in the case of abandoned mine workings, on the plane corresponding to the dip of the ore body, whichever will show the passages to the best advantage. The extent of stopes and the longitudinal elevations of drifts are shown to good advantage in such views.

94. Perspective Drawings

The maps and views discussed in paragraphs 89 through 93 have the disadvantage of showing only a two-dimensional view of the passageways. The three-dimensional aspect of the underground workings is visualized more readily by reference to perspective drawings. These portray the underground conditions nearly as well as models and are much more quickly prepared.

95. Special Plans

Special plans and elevations are drawn to show ventilation systems, the locations of fire-fighting and rescue equipment, the details of stopes, and for similar purposes.

96. Appropriate Scales

To properly show all pertinent details, the various maps and views are commonly drawn on a scale of 1 inch = 40 feet or 1 inch = 50 feet. Where little detail need be shown, scales from 1 inch = 100 feet to 1 inch = 400 feet are satisfactory.

97. Details To Be Plotted

Details to be plotted include property lines, shafts, drifts, stopes, rooms, winzes, raises, crosscuts, chutes, openings in caves, and control stations. Large-scale drawings may be required to correctly portray winding passageways.

98. Finishing the Map

The finished map must show all pertinent details, coordinate lines, an appropriate title (usually in the lower right-hand corner) including the name of the workings or section thereof, the scale, the true and magnetic meridians, the names of the surveyor and draftsman, the date of the survey, and an appropriate border.

99. Office Records and Filing

The office records of underground surveys comprise a file of all survey field notes, a list of all horizontal and vertical control points with their coordinates and location descriptions, and a complete file of all maps and drawings. Original and duplicate drawings of convenient size should be filed flat in a map case. Drawings prepared on long rolls should be filed in a compartment file and should be provided with a title on the reverse side at each end of the roll so that the drawing can be readily identified without unrolling. Maps and drawings which are in frequent use should have cloth or heavy paper bindings stitched or pasted on their edges to prevent tearing of edges or rolling or folding of the corners. The office records may also include three-dimensional models of the underground passages.

100. Models

a. Three-dimensional models of underground passageways are useful in military operations and have application to underground engineering projects. They aid materially in the visualization of the relationships of the various openings. Both models and maps are useful for showing the progress of underground construction or for showing the military utilization of underground workings.

b. The types of models commonly used are classified as skeleton models, vein models, timbering models, solid models, and plastic or glass-plate models.

c. Of the model types listed in *b* above, the skeleton model is the simplest to construct. It consists of a scale reproduction of the shafts, drifts, and other workings supported, in their correct relative positions, above a wooden base by a light-metal or wooden frame. A scale of 1 inch = 40 feet is common. If the scale is much smaller, it is difficult to show details; if much larger, the model becomes unwieldy.

Coordinate lines of the grid system used are shown on the wooden base. The models of the workings are usually of wood, roughed out with a jigsaw and smoothed to shape with chisel, knife, and sandpaper. Relatively regular workings may be shaped from hardwood for greater durability but soft pine is generally used for the material from which the shapes of irregular stopes are cut. The parts of the model are joined together and to the supporting trusses of framework by brads, wooden pegs, or glue. The workings are painted in one color, the supporting members in another. Labels to show the names of the various features are lettered and glued to the model in appropriate locations. Surface topography can be represented by bent wires, each indicating a contour and each being fastened at intervals to the supporting frame or the contour wires above and below it. Sheets of screen wire, sponge rubber, or plastic material, shaped to conform to the topography, are even better.

d. Other type models are more elaborate and require special materials and the services of experienced model-building craftsmen. They will be produced by such trained technicians if required for planning or operational purposes.

CHAPTER 3

SURVEY SUPPORT OF GEOLOGY AND PEDOLOGY

Section I. GEOLOGY

101. Introduction

The end product of most topographic surveys is a topographic map. In geology or other related sciences the topographic survey is the first part of a series of interrelated surveys, the end product of which is a map containing not only topographic information but also other specialized data keyed to it. In geologic investigations, commonly referred to as surveys, a geologist makes systematic observations of the physical characteristics, distribution, geologic age, and structure of the rocks as well as the ground water and mineral resources that the rocks contain. These observations are expressed in finished form as geologic maps and texts. The objective of the geologist is to portray, in plan or section, geologic data required for subsequent military use.

102. Military Application

Pure geologic data has little direct application to military problems. However, if the field information is interpreted into specialized lines it is of considerable use in military planning and operations. Military requirements may necessitate regional geologic study and mapping, surveys of more limited areas, or the development of detailed geologic data at a construction site.

103. Methods of Geologic Surveying

Most geologic data are gathered from an examination of rocks in the field. In addition, examination of drainage and relief patterns on detailed maps or aerial photographs provides considerable supplementary data on rock structures and distribution.

a. Conduct of Survey. In the field, the geologist conducts his survey by examining the rock wherever it is exposed at the surface and not covered by soil or other material. At such exposures, called outcrops, he systematically records the physical characteristics of the rock, thickness of exposure, inclination of rock bedding, and development of joints or fractures. In addition, the age of the rock is determined from fossils or the sequence of rock units. Rock investigations are not confined to surface exposures, as the deeper seated rocks are examined by using samples obtained from auger or boreholes. The information gathered by the geologist is placed on a map base by plotting the rock types in color with other data incorporated as symbols or annotations. To amplify the map data, more complete descriptions of outcrops are entered in notebooks with the entries keyed to the field map. Survey troops support the geologist by preparing the basic topographic maps on which the results of geologic investigations are plotted and by making such tie measurements to geologic features as the geologist may require.

b. Plotting Methods. The geologist uses simple survey methods in plotting geologic features on a field map. Where an outcrop can be located with reference to a cultural or relief feature, it is generally plotted on the map by spot recognition. In other cases, the relation of a geologic feature to a recognizable topographic feature is established by using a Brunton compass to determine direction, and by pacing or taping to measure distance. Slope or small differences in elevation are measured by using a clinometer or hand level, while an altimeter is used where there are large differences in elevation. When the geologic survey is keyed to a large-scale plan, the geologist generally uses a planetable and data are plotted with accuracy commensurate with the accuracy of the base plan.

104. Base Maps for Reconnaissance Geologic Surveys

a. Reconnaissance geologic mapping is exploratory in nature. Large areas are covered and the detail obtained gives a general picture of the geology of the region, plus information for determining areas suitable for more precise surveys. For reconnaissance surveys, base maps at scales smaller than 1 inch to the mile are generally used. No special surveys for a base are executed if the area involved in the geologic reconnaissance is covered by satisfactory maps already published. If no base maps exist, a hasty planimetric survey will provide a satisfactory base. In general, the ground survey for such a base map covers the entire area of the reconnaissance survey except where the geologist confines his reconnaissance to isolated line of traverse. In this case the survey for the base is generally limited to the area including and immediately adjacent to the lines of traverse.

b. The scale and contour interval of maps covering an area of several square miles or square kilometers will be specified by the geologist to meet the requirements of his objective. Normally, a scale of 1:25,000 will be suitable. The contour interval may be as small as 5 feet for flat relief, 10 feet in moderately hilly country, and 20 feet or more where the terrain is rugged. Basic control, both horizontal and vertical, should extend over the area with located points not more than 3 miles or about 5 km apart. Such control stations may have been established by previous surveys. If not, control surveys (TM 5–441) will be required prior to topographic mapping of the area.

105. The Base Map Survey

a. The survey for the base map should precede the geologic survey, because the geologist uses the map in the field to plot his data and to determine his position by identification of topographic details. Where aerial photographs are available, the base map need not be made before the geologic survey, since the geologist can use the aerial photograph as a plotting base and later transfer the data to a map base. However, where possible, the base should be prepared in advance as the number of aerial photographs needed to cover an area is generally too large to be handled in the field.

b. Planetable topography is the method best suited to relatively open country. In the absence of detailed instructions, the following specifications are generally satisfactory.

(1) *Base direction.* To determine a base direction, take from a known base, a side in a triangulation net, or a course of a basic control traverse.

(2) *Local horizontal control.* Use planetable traverses run in closed circuits or between known control stations of a higher order of accuracy, or locate planetable stations by graphical triangulation.

(3) *Local vertical control.* Where the terrain is relatively level, carry elevation along traverses by vertical angle or stadia-arc measurements, adjusting elevations on closure at a basic control station. For rugged terrain mapped at one of the larger contour intervals, barometric or trigonometric leveling is suitable.

(4) *Sights.* Use telescopic alidade.

(5) *Distance measurements.* Use, in general, stadia or graphical triangulation to locate points and stations. Certain measurements can be made most conveniently by pacing or rough taping.

(6) *Contouring.* Locate and determine the elevations of controlling points on summits, in valleys and saddles, and at points of marked change of slope. Interpolate and sketch contours *in the field*, using these elevations for control.

(7) *Accuracy.* Distance measurements by stadia should be accurate to 1 part in 500. Side-shot points located by pacing or other rough measurements should be accurate to within 25 feet (7.6 m). Sights for traverse lines or graphical triangulation should be taken with care to obtain the maximum accuracy inherent in the telescopic alidade. The error in the elevation of any point, as read from the finished map, should not exceed one-half the contour interval.

c. Topography may be located more conveniently in heavily timbered country by stadia measurements from transit-stadia traverse than by the use of the planetable, although the time required for plotting will be increased. The specifications listed in *b* above are generally applicable. Read horizontal angles on traverses to 1 minute, and horizontal angles for side shots which will be plotted by protractor to the nearest quarter-degree. Read vertical angles for elevation determination to 1 minute or use the stadia arc. Keep complete and carefully prepared stadia notes and sketches to assure correct plotting.

d. When the geologist indicates that a map of a lower order of accuracy will fulfill his needs, planetable or compass traverses as specified for pedological surveys (paras. 116 and 117) are suitable.

106. Accuracy of Position

The accuracy of position of a reconnaissance geological survey is generally low. The survey for the base map can be of a correspondingly low accuracy. Horizontal and vertical distances with an accuracy of 1:100 and angles measured to within 1° are satisfactory. Adjustment of errors in closure can be made graphically. Since the survey is of relatively low accuracy, compass and tape traverses are adequate; but where a large area is involved, longer sights obtained by transit or planetable methods may be more expeditious. Elevations based on altimeter observations or determined from vertical angles should be numerous so that the geologist will be able to tie in his vertical measurements without resorting to long lines of levels. To satisfy this need, elevations should be determined for all important landmark features such as road intersections, summits of peaks or ridges, and stream junctions.

107. Use of Aerial Photographs

Where aerial photographs are available, the geologist generally uses them in the field in lieu of a map. The most satisfactory results are obtained from large-scale photographs 1:15,000 or larger. Some topographic features, such as some ravines, rocky knobs, or sinkholes, are too small to be shown on maps. These features, as well as the larger topographic forms such as stream channels and swamps, can be observed directly from aerial photographs. The photos also can be used to prepare a base map for portrayal of the field data by tracing planimetric detail from an uncontrolled mosaic with spot elevations added from field surveys. Use of contact prints of aerial photographs by the geologist in place of a map base is satisfactory, except where large-scale plans for engineering purposes are to be the base. In such a case the distortion within an aerial photograph does not permit plotting of geologic data commensurate with the accuracy of the final plan.

108. Map Bases For Detailed Geologic Surveys

The detailed geologic survey generally covers a specific map area, geographic region, or specified site from scales of 1:62,500 to those of 1:600 or larger. In general, the very large scales are used for specific engineering or mineral development problems.

a. Site Plans and Profiles. Geologic data affecting foundation design at construction sites are plotted on plans drawn to scales of 1 inch = 50, 100, 200, or 400 feet. Contour intervals may range from 1 to 10 feet, depending upon the roughness of the terrain. Planetable mapping is suited to plotting the topographic features, ranges, and reference points used to locate drill holes, rock outcrops, and other geologic data. When plotting contours on a 1- or 2-foot interval it is better to locate points which are actually on the contours or to determine elevations at the intersection of closely spaced grid lines staked out on the site rather than to use the method of contouring as cited in paragraph 105*b*(6). In addition to a plan, the geologist may require that profiles be drawn along selected lines or that the logs of drill holes be plotted to suitable scales.

b. Using a Topographic Map as a Base Map. The base map for a detailed geologic survey is a complete topographic map or plan with relief expressed by contours. Colors and symbolization of basic detail are simple so that they will not conflict with the overlay of geologic information that is shown by colors and symbols. Published topographic maps are used where suitable. The geologic survey is

expedited if the map base is from a quarter to double the scale of the map on which the information is to be presented. Enlargements of the base map are generally used to satisfy this requirement, rather than using other maps of a larger scale. This permits the direct reduction of geologic data to the scale of the final map with a minimum amount of drafting.

c. When No Topographic Map is Available. If existing maps are not suitable, a base map or plan must be prepared from detailed topographic surveys. Culture and relief (contours) should be shown in the greatest detail possible. The survey for the base should conform to third-order accuracy where large geographic areas are concerned. Maps made from aerial photographs using precise instrument methods, such as multiplex, can be used in place of field surveys. Altitude or elevation of the intersection of boreholes and the surface should be accurate to the nearest half-foot.

109. Field Sheets

a. Finishing. Upon completion of the fieldwork, the survey engineer furnishes the geologist with clean presentable maps, overlays, and sketches for his plotting of geologic data. The plots of the field survey should be adjusted for closure, contain the necessary marginal data, and fulfill the accuracy requirements prescribed by the geologist.

b. Adjusting Closure. Closure adjustments are made in accordance with the assumption that the error is accumulated uniformly or in proportion to the distance traversed. The correction for closure at intermediate traverse stations is applied by moving the station in a direction parallel to that indicated by the closing error. The amount of correction applied at an intermediate station should bear the same ratio to the traverse closure error as the distance traversed to the intermediate station bears to the total traverse length. Closure adjustments usually are made by eye. If a high degree of accuracy is required, the adjustment is made either analytically or graphically.

c. Symbols and Data. Geologic symbols and data are plotted by the geologist, from the measurements and notes, on the base map prepared by the surveyor.

Section II. PEDOLOGY

110. Introduction

There is a military requirement for pedological mapping for the purpose of locating the limits of sand or gravel deposits suitable for concrete aggregates, road materials, or construction operations. The pedological survey is conducted under the direction of the soils engineer, and the surveyor's mission is one of support to the soils engineer's objective.

111. References

TM 5–545 provides a reference covering basic geologic terms and TM 5–541, covers the unified soil classification system. Reference should also be made to TM 5–230 for methods of drafting and sketching.

112. Equipment

a. The military sketching set, surveying, SM 5–4–6675–S30 or survey set, plane table, SM 5–4–6675–S38 provide the basic surveying instruments.

b. The test set, soil, SM 5–4–6630–S02, provides the basic soil testing tools for sampling soils.

c. Drilling machines (earth boring) TM 5–3820–208–15, and Drills Pneumatic, SM ENG 7 & 8–4089 provide the necessary equipment and tools for drilling test holes.

113. Objective

The engineer's objective in a pedological survey is to prepare data in plan and profile symbolizing soils and outcroppings on maps, overlays, and sketches for subsequent military usage. The following paragraphs pertain to the survey support needed in the preparation of such maps and sketches and to prescribe the methods for performing the necessary survey measurements.

114. Application

The soils survey has three general types of application—

a. Aerial photography is used when an extensive area is to be surveyed. Usually there are no survey measurements required in this case.

b. Maps of an area of several square miles in extent are required when an initial study or technical reconnaissance is needed for an engineering project. Low-order survey measurements usually suffice for the preparation of a reconnaissance sketch upon which the soils engineer can plot the pertinent data.

c. A sketch of an airfield site is frequently required by the soils analysts before construction planning can be initiated. In this case the surveyor applies low-order measurements to prepare a sketch (scale 1 inch = 100, 200, or 400 feet) upon which the soils engineer plot the results of soil tests and findings.

115. Aerial Photography

Photo coverage of the area under consideration facilitates the establishment of control for a pedological survey. The use of vertical aerial photographs in the planning phase of outlining ground control will speed the survey regardless of the size of the area to be covered. If controlled photographs are available, the survey engineer can locate points by pricking or keying them to the photographs. An uncontrolled photograph may be satisfactory for the surveys of low-order accuracy mentioned in the preceding paragraph. The survey engineer prepares, according to the soils analysts instructions, maps or overlays upon which are plotted the control and ties to pedological features. The pedological interpretation of aerial photographs is the responsibility of the terrain analysts.

116. Planetable Traverse

The planetable traverse is best adapted to relatively open country for the preparation of the basic sketch upon which the soils engineer plots pertinent data. In the absence of detailed instructions from the soils engineer, the following procedures are generally satisfactory for preparing a sketch of an area of several square miles (3 miles by 3 miles or 4.5 km by 4.5 km maximum for initial exploration).

a. Scale. 1:12,500 or 1:25,000.

b. Traverse Control. Run in circuits or between known positions of a higher order of accuracy.

c. Sighting. Use a boxwood or a telescopic alidade.

d. Distance Measurements. Pace or rough tape. When telescopic alidade is available, use stadia measurements where possible with a view to reducing the time required for the survey rather than increasing the accuracy.

e. Base Direction. To determine a base direction select known bases, railroad, or highway tangents, or features recognizable or reliable topographic maps; otherwise, use magnetic north as determined by compass observations.

f. Compass. Use military compass, forestry compass, or pocket transit.

g. Distance Between Basic Control Points. Maintain 3 miles (4.8 km) as the extreme maximum distance between stations.

h. Accuracy. Distances should be measured in such a manner that points can be plotted within 25 feet (7.5 m). For the scales suggested, measurements to 1 part in 100 will suffice. Take sights with boxwood alidade with care to maintain directions of an accuracy comparable to distances.

i. Topography. Usually not required on reconnaissance surveys for pedology, particularly in areas of low relief. Where suitable deposits of sand, gravel, or stone have been located, route surveys from the site to the point of use may be required for the location of haulage roads, conveyors, or other means of transporting the material. In hilly terrain, rough topography, obtained by clinometer, pocket transit, or stadia, may be required to facilitate the location of a favorable route.

117. Compass Traverse

The compass traverse is more convenient in heavily wooded areas although more time is required for plotting than is the case with planetable traversing. Traverse lines between stations should be long in order to reduce the number of observed bearings. Points between stations are located by offsets from the traverse lines. Where local attraction affects compass readings, points are plotted by intersection.

Survey readings may be plotted in the field. Notes should be kept in case it is necessary to retrace the traverse. In the absence of detailed instructions from the soils engineer, the basic guides listed in paragraph 116 apply.

118. Field Sheets

The survey engineer must furnish the soils analysts with suitable maps, overlays, and sketches for the plotting of pedological data. The information given in paragraph 109 concerning field sheets for the geologist will apply.

119. Site Plans

After the preparation of a reconnaissance field sheet of an area of several square miles, the soils analysts may require a sketch of a particular site in which many samples are taken for a more detailed study as mentioned in paragraph 114*c*. In the absence of detailed instructions, the surveyor prepares a sketch on a scale of 1 inch = 400 feet and provides ranges and reference points to aid in plotting or tying in specific positions of auger holes, drill holes, and lines of exposed rock or other pedological features. For plotting the data of a range, cross section, or series of boreholes, the soils analyst may require the surveyor to provide a basic plot on a scale of 1 inch = 100 feet or of 1 inch = 200 feet. Survey measurements will be conducted accordingly.

CHAPTER 4
LAND SURVEYING

Section I. OPERATIONS

120. Introduction

Land surveying embraces those surveying operations involved in original surveys to locate and monument the boundaries of a property; the preparation of a legal description of the limits of a property and of the area included; the preparation of a property map; resurveys to recover and remonument property corners; and surveys in connection with the subdivision of a property into two or more parts.

121. Military Application

a. In military surveying it is sometimes necessary to retrace surveys of property lines, to reestablish lost or obliterated corners, and to make ties to property lines and corners. For example, a retracement survey of property lines may be required to assure that the military operation of quarry excavation does not encroach on adjacent property where excavation rights have not been obtained. Similarly, an access road from a public highway to the quarry site which crosses privately owned property should be tied to the property lines that are crossed so that correctly executed easements can be obtained to cross the tracts of private property.

b. Military surveyors may be required to accomplish property surveys outside the continental limits of the United States in connection with engineer activities such as the acquisition of property in friendly foreign nations for the construction of military bases, the leasing of land and facilities thereon for military usage, and the restoration of such properties to property owners.

c. Land surveying is one of the oldest branches of engineering known to man, the principles of which have been passed down through the centuries. For this reason, the essentials of land surveying as practiced in various countries are similar in principle. Although the principles pertaining to the surveys of public and private lands within the United States are not necessarily directly applicable to foreign areas, a knowledge of these principles will indirectly assist the military surveyor to analyze the survey practices of a foreign nation and thereby assist him to conduct the survey in a manner required by the property laws of the nation concerned.

122. Forms of Land Description

A parcel of land may be described by metes and bounds; by stating its location and size in a rectangular system of land subdivision; by giving the coordinates of the property corners with reference to a plane coordinate system; by a dead reference to a description in a previously recorded deed; or by reference to block and individual property numbers appearing on a recorded map.

123. Metes and Bounds

When a tract of land is defined by giving the courses and lengths of all boundaries it is said to be described by *metes and bounds.* This is an age-old method of describing land and still forms the basis for the majority of deed descriptions in the eastern states and in many foreign lands. A good metes and bounds description starts at a point of beginning which should be monumented and referenced by ties or distances from well established monuments or other reference points. The bearing and length of each side is given in turn around the tract to close back on the point of beginning. Bearing may be true or magnetic grid, preferably the former. When magnetic bearings are read, the declination of the needle

and the date of the survey should be stated. The stakes or monuments placed at each corner should be described to aid in their recovery in the future. Ties from corner monuments to witness points (trees, poles, boulders, ledges, or other semipermanent or permanent objects) are always helpful in relocating corners, particularly where the corner markers themselves lack permanence. In timbered country, blazes on trees on or adjacent to a boundary line are most useful in reestablishing the line at a future date. It is also advisable to state the names of abutting property owners along the several sides of the tract being described. Many metes and bounds descriptions fail to include all of these particulars and are frequently very difficult to retrace or locate in relation to adjoining ownerships.

124. The Rectangular System

In the early days of the United States, provisions were made to subdivide territorial lands into townships and sections thereof, along lines running with the cardinal directions of north-south, east-west. Later, as additional lands were added to the public domain, such lands were subdivided in a similar manner.

125. Plane Coordinate Systems

For many years the triangulation and traverse monuments of various domestic and foreign survey agencies have been defined by their geographic positions, that is, by their latitudes and longitudes. Property corners might be definitely fixed in position in the same way. The necessary computations are involved and too few land surveyors are sufficiently well versed in the theory of geodetic surveying for this method to attain widespread use. In recent years, plane coordinate systems have been developed and used in many states of the United States and in many foreign countries. These grid systems involve relatively simple calculations and their use in describing parcels of land is increasing. A plane coordinate system based on the Lambert Conformal projection is used in those states extending generally east and west, while a system based on the Transverse Mercator projection is employed in those states having their greatest length in a north and south direction. In the larger states, several plane coordinate systems may be used, each applying to a portion thereof, so that the errors inherent in any plane coordinate system applied to a spheroidal surface may be kept within narrow limits. For descriptions of plane coordinate systems that are used in the several states see U.S. Coast and Geodetic Survey Special Publications 235, *The State Coordinate Systems*, and 193, *Manual of Plane Coordinate Computation*. Similar plane coordinate systems are used in foreign countries. Once the coordinates of the corners of a property have been established, the property boundaries can be relocated even though all corner monuments have been destroyed. Such systems have been of the utmost value in reestablishing property lines in war-ravaged countries in Europe.

126. Relations of Geodetic Surveys to Land Surveying

Triangulation, precise traverse, and other geodetic or astronomic control surveys are required to fix the base lines and principal meridians used in the rectangular system of land surveys and to provide control points for extending the coordinates of plane coordinate systems to local land surveys. In many areas, the stations of the triangulation network have been connected by geodetic control traverses running along the major streets, highways, and railways. Pairs of control monuments are set at frequent intervals so that it is usually only a short distance to the site of a local property survey. The geographic position of the monuments and their elevation are available in the public records. Methods of executing the necessary geodetic control surveys are given in TM 5–441.

127. Difficulties Encountered in Resurveys of Old Boundaries

The relocation of property boundaries frequently presents difficulties arising from indefinite points of beginning; indefinite meridians; failure of distances to check; obliterated corners; and inability to traverse along boundaries.

a. Point of Beginning Indefinite or Difficult to Locate. The following are phrases extracted from old deed descriptions: "Beginning at a

drill hole in a ledge on the easterly shore of Lake Champlain . . ."; "beginning at the point where you and I stood talking yesterday . . ."; "beginning at the northwest corner of Dana's upper pasture." Lake Champlain is over 100 miles long and it might well prove time consuming to locate the drill hole and ledge referred to. The second phrase is meaningless except to the two parties to the original transaction. They may be long since dead or, if still living, uncertain in their recollection of the location of the point in question. The upper pasture may have grown up to brush and forest years ago and all trace of its corners obliterated. To resolve such difficulties, the surveyor must obtain information from old residents whose opinions may be biased or whose memories may be faulty, he must scan aerial photographs for evidence of old lines of cultivation or logging, and he must make a thorough search of the locality for monuments, line blazes, or witness trees.

b. Indefinite Meridian. Some deed descriptions give bearings of the property lines but fail to indicate whether these are true or magnetic or to state the magnetic declination or date of survey. It may not be possible to resolve this difficulty until two adjacent monuments have been recovered. Then the true bearing of the line between them can be determined and compared with the bearing given in the deed and a correction angle applied to all other bearings in the description as an aid in searching for other corner monuments.

c. Failure of Distances to Check. The lines in many old surveys were measured with inaccuracy or with low precision. Low property values did not require very careful work. Distances were sometimes measured by counting the revolutions of a wagon wheel and the tally was often incorrect. Distances measured with a chain were frequently in error because of kinks in the link wires, because of wear on the links, or because of poor alinement or plumbing. Many early surveyors made a practice of giving full measure so that the distances given in the deed description are frequently less than those measured in the resurvey. It is not the function of the surveyor to correct errors in the original survey but rather to locate the corners as originally established. When two adjacent corners have been relocated, the taped distance between them can be compared with the distance as given in the deed. The other deed distances are then adjusted in proportion to give lengths which will aid in locating the remaining corner monuments.

d. Obliterated Corners. When the original property corner consisted of a tree- a wooden stake, or an iron pipe, rotting of the wood or rusting of the iron may have destroyed the corner. Measurements of the bearing and distance from an adjacent corner will give an approximate location. Careful removal of the surface litter in this area may then disclose evidence of the rotted stake, stump, or pipe. The humus from the wood and the rust from the pipe will generally show a distinct difference in coloration and physical characteristics in contrast to the surrounding soil. Four stakes can then be driven nearby in such positions that cords stretched between diagonally opposite stakes will intersect over the position of the original point. A hole can then be dug at this point and a permanent concrete or stone monument installed, the monument being centered under the intersecting cords.

e. Inability to Traverse Along Boundaries. The actual boundaries of a property may be occupied by fences, stone wall, hedgerows, or a line of shade trees. Adjacent corners may not be intervisible. Such conditions necessitate the use of offset lines or random lines and add to the difficulty of conducting the survey.

128. Laws Relating to Boundaries

The body of laws and precedents relating to boundaries is voluminous. A complete coverage of these various laws would fill several volumes. The land surveyor has no judicial function but, if he is to perform his work skillfully and well, he should have a knowledge of the property laws and rulings of the locality in which he is serving. In this manual it is only possible to set forth a few guiding principles and to indicate a few important differences in existing laws.

a. It is the function of the surveyor to recover, if possible, the original boundaries and corners of a property. It is not his function to correct errors in the original survey.

b. When an original survey is incomplete or contains errors, it is assumed that it was the intention of the grantor to transfer a definite tract of land to the grantee and the deed will be so interpreted as to make it effectual rather than void.

c. Calls in deed descriptions refer to monuments, distances, directions, adjoining owners, and the area of the tract. When all corners or boundaries cannot be relocated or retraced with certainty, the greatest weight must be given in the resurvey to those calls which are most likely to be correct and which therefore take precedence over other calls. The usual order of calls in order of their decreasing precedence is as follows:

(1) Natural monuments and boundaries.
(2) Artificial monuments.
(3) Calls for adjoiners.
(4) Courses and distances.
(5) Area.

An exception to the general rule occurs when a will states that equal or other specified divisions of land to several heirs are intended, in which case the call for area, (5) above, will take precedence over the call for course and distance, (4) above.

d. Local usage and rulings vary with respect to where the actual boundary lies within a fence, party wall, hedge, ditch, or balk. The surveyor should consult local authorities in this matter if the actual corner monuments cannot be uncovered.

e. Where property abuts on a highway, ownership in some localities extends to the centerline of the road, the public enjoying the use of the roadway as an easement and the land reverting to the abutting owners if the roadway is abandoned. In other localities, title to the roadway reside in the state, county, or municipality.

f. The rights of owners of land abutting on the ocean, lakes, or tidal or upland streams or rivers are subject to statutes which vary in different localities. Ownership may extend to the high water mark, to low water, or to the thread or middle of the channel. The owner of such property may have valuable riparian rights in a waterway or may acquire a riparian grant. Local laws must be consulted.

g. Land belonging to another may be acquired after a definite period of time by *adverse possession*. If a property owner *A* occupies not only his own land but also a part of the land of *B*, and if such occupancy has been open, hostile, notorious, and continues for the statutory period, then *A* gains title to that portion of *B's* land so occupied. The statutory period is generally 20 years. In some localities ownership of part of a public highway can be acquired after 40 years of adverse occupancy; in other localities, adverse possession does not operate against public land.

129. Responsibilities Related to Land Surveys

Responsibilities for supervision of surveys of all types have been itemized in paragraph 3. Additional responsibilities particularly applicable to land surveys include—

a. Obtain permission for survey personnel to trespass on private lands.

b. Where an engineer officer has responsibility for property surveys conducted within the United States, secure the services of personnel to conduct the surveys who are licensed to practice land surveying in the state in question.

c. Conform to the laws respecting surveys of friendly foreign nations in which military property surveys are being conducted.

d. Secure deed descriptions and copies of filed maps covering the property to be surveyed from the local registry of deeds, hall of records, or other repository of public records pertaining to property.

e. Arrange for reasonable compensation to property owners for any damage resulting from the survey operations.

f. Where property is being acquired for military use in the United States, initiate title searches to establish validity of title and any easements or clouds on the title which may exist.

g. Obtain descriptions of the location and coordinates of control monuments in the vicinity.

h. Secure competent legal representation with respect to the legal aspects of property surveys in either the United States or in friendly foreign nations.

i. Secure the cooperation of local surveyors or of other agencies who have conducted surveys in the area and who may be able to supply information and assistance.

Section II. PUBLIC LANDS

130. The Public Domain

At the time of the establishment of the United States, the thirteen original States retained title to all unappropriated lands within their respective boundaries and retained control of surveys of these lands. Later, such title was also retained by the States of Maine, Vermont, West Virginia, Kentucky, Tennessee, and Texas. As successive territories on the North American continent were purchased by or ceded to the Federal Union, title to vacant lands in such territories was vested in the United States. This region (fig. 55) became the public domain. To provide for the subdivision of these public lands and their subsequent settlement, the Congress passed legislation to establish the rectangular system of surveys (para. 124) in the area and to set up procedures for the transfer of land to private ownership. The original legislation, together with revisions enacted by the Congress in later years, forms the basis for the system of surveys used to partition the public lands. A knowledge of the methods which governed the original surveys is essential when resurveys of property are made in those states created from the public domain. Such knowledge may also aid the military surveyor in analyzing the survey practices of foreign nations.

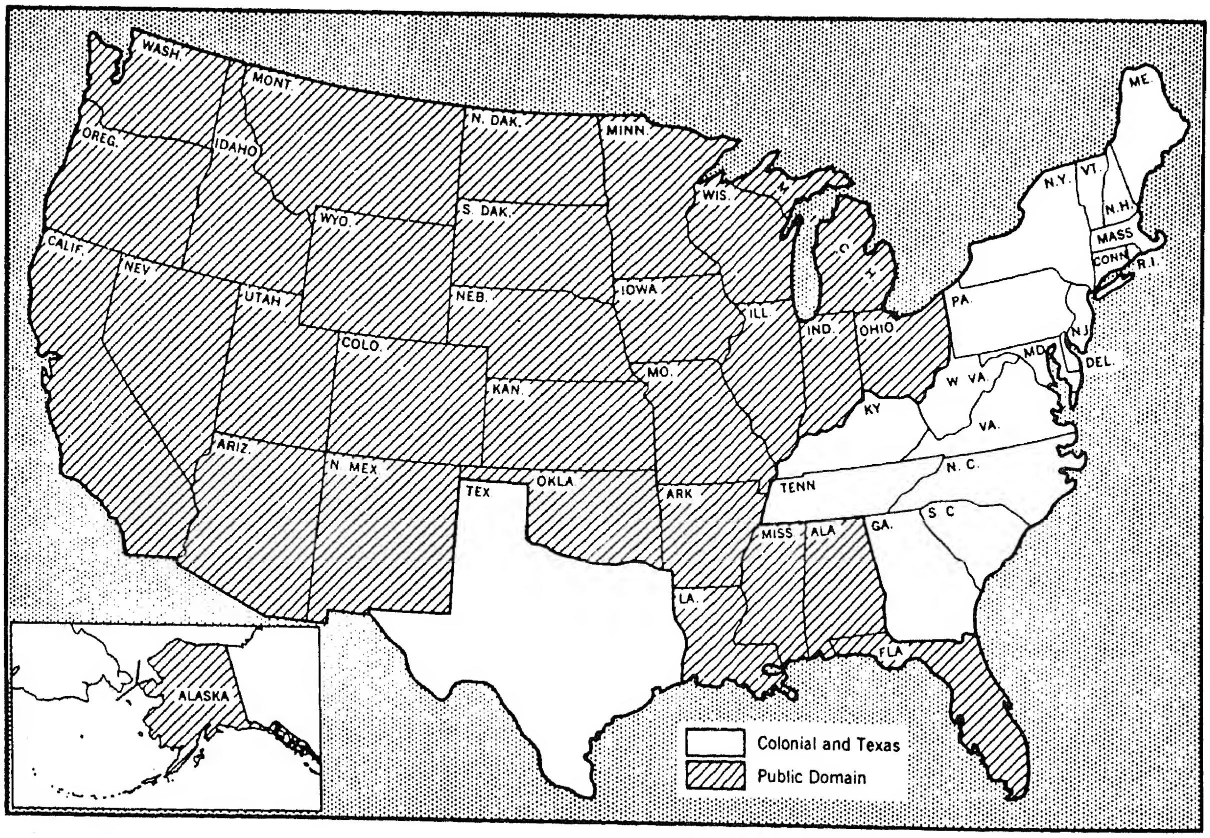


Figure 55. The public domain.

131. General Description of the U.S. System

The legislation provides that the public lands shall be divided by true meridional lines and by parallels of latitude to form townships approximately 6 miles square (9.8 km sq.). The corners of the townships are marked with progressive numbers from the point of beginning. Each township is further subdivided into sections of approximately 640 acres each by lines running generally parallel to the township boundaries. The sections are numbered, beginning with number 1 in the northeast section of the township and proceeding west and east alternately through the township until all thirty-six sections have been numbered. Lands within an Indian reservation and lands previously transferred to private ownership are not included in the subdivision survey. Section lines are also interrupted by navigable rivers.

132. U.S. Bureau of Land Management

Surveys of the public lands come under the jurisdiction of the U.S. Bureau of Land Management formerly the General Land Office. The director of this bureau exercises general supervision of surveys of the public lands and of the disposal of such lands. Seven field regions have been set up, each under the supervision of a regional cadastral domain. Resurveys of certain types are made by cadastral engineers in the service of the Bureau.

133. Need for Knowledge of Methods Used in Public Land Surveys

To properly reestablish the boundaries of a tract of land, the surveyor must be somewhat familar with the methods and procedures used in the original survey. It should be also noted that, when a section of land in a state created from the public domain is subdivided into a large number of small parcels, metes and bounds may be used in describing such parcels. Resurveys in such areas may involve the use of methods covered in paragraphs 167 through 174 as well as the procedures in paragraphs 135 through 149.

134. The U.S. System of Surveying the Public Lands

This system is described in detail in the Manual of Instructions for the Survey of the Public Lands of the United States, edition of 1947, published by the Bureau of Land Management and available from the Superintendent of Documents, Washington, D.C. The major features of the system and current field methods are described in paragraphs 135 through 149.

135. Initial Points

The area embraced by the public domain has been divided into some thirty-five regions for survey purposes. In each region, an *initial point* has been established and monumented, this initial point serving as the origin for public-land surveys to be extended throughout that region. The geographic positions of the various initial points have been determined by ties to geodetic stations.

136. Principal Lines

The surveys are run on cardinal lines, that is, lines running north and south or east and west. All principal lines in the system are therefore meridians of longitude, running north and south, and parallels of latitude, running east and west. The meridians are straight lines but they are not parallel since they converge to meet at the poles. The parallels of latitude are at right angles to the meridian at any point. They are, therefore, curved lines.

137. Base Line and Principal Meridian

Through each initial point two cardinal lines are run. The *base line* is extended from the initial monument east and west on a true parallel of latitude. The *principal meridian* is run as a true north-south line extended from the initial point either north or south, or in both directions, as required by the position of the initial point within the area to be surveyed. The various initial points, base lines, and principal meridians (table II) are designated by a name assigned to the principal meridian.

Table II. Meridians and Base Lines of U. S. Rectangular Surveys.

Meridians	Governing surveys (wholly or in part) in States of—	Longitude of initial points west from Greenwich			Latitude of initial points		
		°	′	″	°	′	″
Black Hills	South Dakota	104	03	16	43	59	44
Boise	Idaho	116	23	35	43	22	21
Chickasaw	Mississippi	89	14	47	35	01	58
Choctaw	do	90	14	41	31	52	32
Cimarron	Oklahoma	103	00	07	36	30	05
Copper River	Alaska	145	18	13	61	49	21
Fairbanks	do	147	38	26	64	51	50
Fifth Principal	Arkansas, Iowa, Minnesota, Missouri, North Dakota, and South Dakota.	91	03	07	34	38	45
First Principal	Ohio and Indiana	84	48	11	40	59	22
Fourth Principal	Illinois [1]	90	27	11	40	00	50
Do	Minnesota and Wisconsin	90	25	37	42	30	27
Gila and Salt River	Arizona	112	18	19	33	22	38
Humboldt	California	124	07	10	40	25	02
Huntsville	Alabama and Mississippi	86	34	16	34	59	27
Indian	Oklahoma	97	14	49	34	29	32
Louisiana	Louisiana	92	24	55	31	00	31
Michigan	Michigan and Ohio	84	21	53	42	25	28
Mount Diablo	California and Nevada	121	54	47	37	52	54
Navajo	Arizona	108	31	59	35	44	56
New Mexico Principal	Colorado and New Mexico	106	53	12	34	15	35
Principal	Montana	111	39	33	45	47	13
Salt Lake	Utah	111	53	27	40	46	11
San Bernardino	California	116	55	17	34	07	20
Second Principal	Illinois and Indiana	86	27	21	38	28	14
Seward	Alaska	149	21	24	60	07	36
Sixth Principal	Colorado, Kansas, Nebraska, South Dakota, and Wyoming.	97	22	08	40	00	07
St. Helena	Louisiana	91	09	36	30	59	56
St. Stephens	Alabama and Mississippi	88	01	20	30	59	51
Tallahassee	Florida and Alabama	84	16	38	30	26	03
Third Principal	Illinois	89	08	54	38	28	27
Uintah	Utah	109	56	06	40	25	59
Ute	Colorado	108	31	59	39	06	23
Washington	Mississippi	91	09	36	30	59	56
Willamette	Oregon and Washington	122	44	34	45	31	11
Wind River	Wyoming	108	48	49	43	00	41

[1] The numbers are carried to fractional township 29 north in Illinois, and are repeated in Wisconsin, beginning with the south boundary of the State; the range numbers are given in regular order.

138. Distance Measurements Along Cardinal Lines

The unit of distance measurement in the public-land surveys is the surveyor's (or Gunter's) chain of 66 feet. Steel tapes, graduated in links and chains, are used today rather than the older chains. Distances are measured along both the base line and the principal meridian, and monuments are placed at intervals of 40 chains (½ mile) to mark quarter-section or section corners (para. 142, 144). Township corners are placed every 6 miles along these lines. Distances are measured twice and remeasurements are made if the discrepancy between the initial two measurements exceeds a specified limit.

139. Division Into Tracts Approximately 24 Miles Square

The survey area is next divided into tracts approximately 24 miles square in the following manner. From monuments placed along the principal meridian at intervals of 24 miles from the initial point, *standard parallels* (also called *correction lines*) are run east and west from the principal meridian. These are monumented at intervals of 40 chains. The standard parallels are designated with respect to their rela-

tionship to the base line and become the First Standard Parallel North, Second Standard Parallel North, First Standard Parallel South, and so on until the limits of the survey area have been reached. *Guide meridians* are then extended *north* from the base line, and from each standard parallel, at intervals of 24 miles east and west from the principal meridian. Each guide meridian is terminated at its intersection with the next standard parallel to the north of its point of origin. The guide meridians become known as the First Guide Meridian East, First Guide Meridian West, and so on, depending upon their locations with respect to the principal meridian. As with the other principal lines, the guide meridians are monumented at intervals of 40 chains. The tract of land bounded by successive pairs of standard parallels and guide meridians will have theoretical boundary lengths of 24 miles on the west, south, and east sides. The north boundary, because of the convergence of the meridians, will be less than 24 miles long. Because of errors of field measurements, the boundaries will seldom equal their theoretical lengths.

140. Division Into Townships

These approximately 24-mile square tracts are next divided into *townships. Range lines* are run north along true meridians from the monuments (known as *standard township corners*) placed at 6-mile intervals, east and west of the principal meridian, along the base line and each standard parallel. As is the case with the guide meridians, each range line terminates at its intersection with the first standard parallel to the north of its point origin. These intersection points, at the termini of range lines and guide meridians, are known as *closing township corners.* Township corners are established at 6-mile intervals on these range lines, and quarter-section and section mounments are placed at intervals of 40 chains. *Township lines,* or latitudinal lines, are then run along parallels of latitude to join the township corners marked by monuments previously set at 6-mile intervals along the principal meridian, guide meridians, and range lines. These range lines and township lines divide the area into townships. The lengths of the east and west boundaries of a township are theoretically

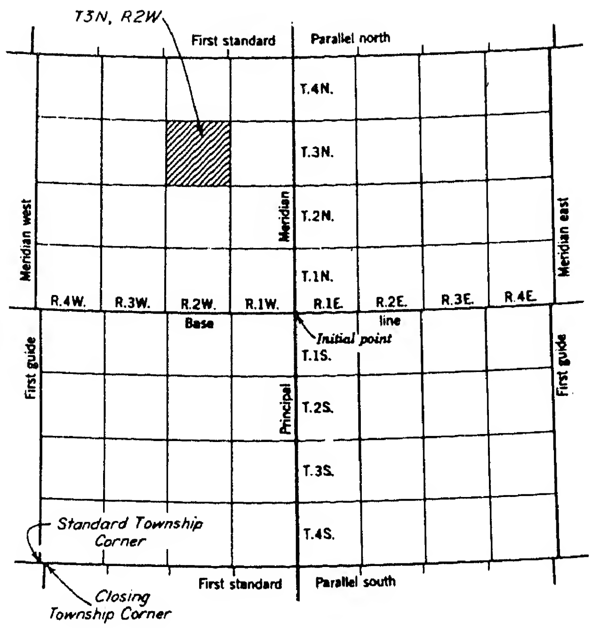


Figure 56. Subdivision of quadrilaterals into townships.

equal to 6 miles. The north and south boundaries of a township vary from a maximum value of 6 miles for its southern boundary at the base line or standard parallel forming the south line of a 24-mile quadrilateral, to a minimum value at its northern boundary. Figure 56 shows the division of 4 such tracts into 16 townships by a base line, principal meridian, standard parallels, guide meridians, range lines, and township lines.

141. System of Numbering Townships

A series of adjacent townships running east and west is known as a *tier.* Such a series of townships running north and south is known as a *range.* The tiers of townships are numbered consecutively, both to the north and south of the base line. The ranges of townships are likewise numbered consecutively, both to the east and west of the principal meridian. A township is designated by the serial number of its tier and the letter *N* or *S* to indicate the position of the tier north or south of the base line, followed by the serial number of its range and the letter *E* or *W* to indicate the position of the range east or west of the principal meridian. Assuming that figure 56 repre-

sents the start of the subdivision along the Fourth Principal Meridian, the cross-hatched township shown in that figure would be designated "Township 3 north, Range 2 west, of the Fourth Principal Meridian." This would be abbreviated "T 3 N. R 2 W, 4th P. M."

142. Division Into Sections

The next step involves the subdivision of each township into sections, each having an area of approximately one square mile or 640 acres. *Meridional section lines* are initiated at the section corners which have been set at intervals of 80 chains (1 mile) along the south boundary of the township and are run from the south to the north boundary parallel to the east boundary of the township. Quartersection and section corners are established alternately at intervals of 40 chains along the meridional section lines. From the section corners so established, *latitudinal section lines* are run from west to east as random lines parallel to the south boundaries of the respective sections to the corresponding section corners on the next meridional section line (or township line) to the east. The falling on the objective corner is noted, and the true line established from east to west. The section lines will divide the township into 36 sections, rhomboidal in shape but nearly square. All sections, with the exception of the six in the most westerly range of the township, will have theoretical dimensions of 80 chains on all four sides. In practical application of this plan, the accumulated error of measurement is placed in the last interval closing on the north boundary of the township. Those sections in the westerly range will be less than a mile in width from east to west, the variation depending upon the distance of the section boundary from the southerly boundary of the 24-mile quadrilateral.

143. Numbering of Sections

The sections are numbered (fig. 57) beginning with 1 in the northeast section of the township and progressing west and east alternately along the tiers of section until all have been numbered.

144. Subdivision of Sections

Sections are usually subdivided into quarter-sections, except for those sections on the westerly range and on the northerly tier of the township. These sections will, in general, contain either more or less than 640 acres because survey errors are accumulated in the northerly 20 chains of each range or meridional section line or because of the convergence of the meridians. Such sections (fig. 58) are commonly subdivided into quarter-sections, half-quarter-sections, quarter-quarter-sections, and numbered fractional lots. Similar special subdivisions are required in the southern tier or in the eastern range of sections when the south boundary (fig. 59) or the east boundary, respectively of the township is defective in alinement. Fractional subdivisions are also required when there is a lake or other navigable body of water within the section or when there are patented mineral claims, private land claims, reservations, state lines, and other special surveys in the area. These subdivisions of section lines are protracted on the official plat of the township by the cadastral engineers of the Bureau of Land Management. Field surveys of these subdivisions of section lines are normally executed by local surveyors in accordance with the protracted lines of the official plat.

145. Marking Corner Monuments

All types of corner monuments and accessories are systematically marked to furnish identification of the monument.

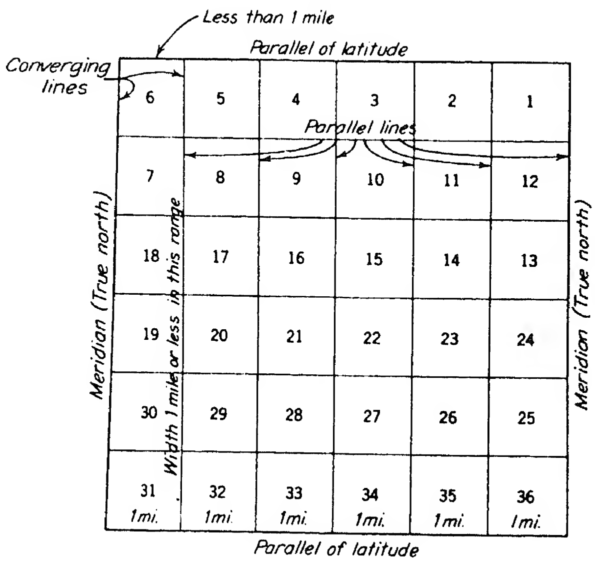
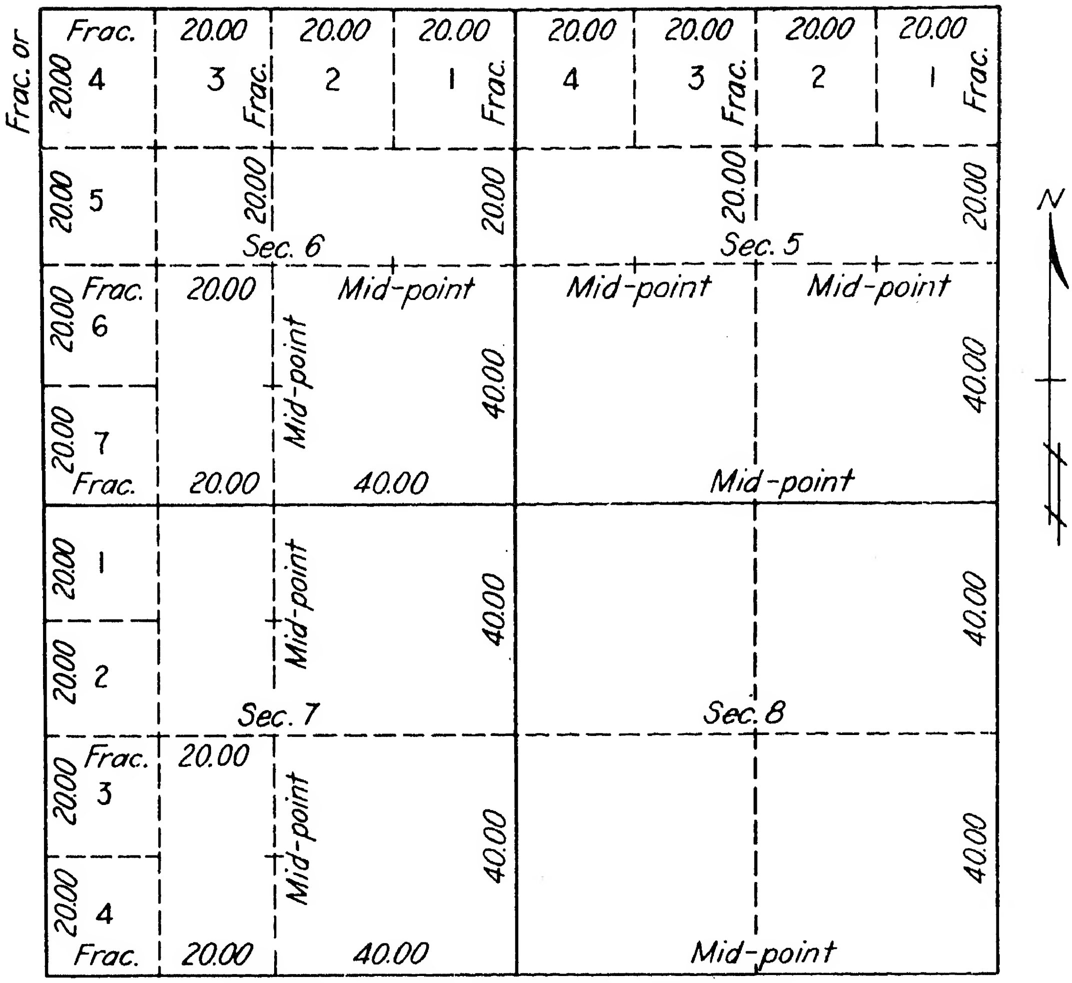
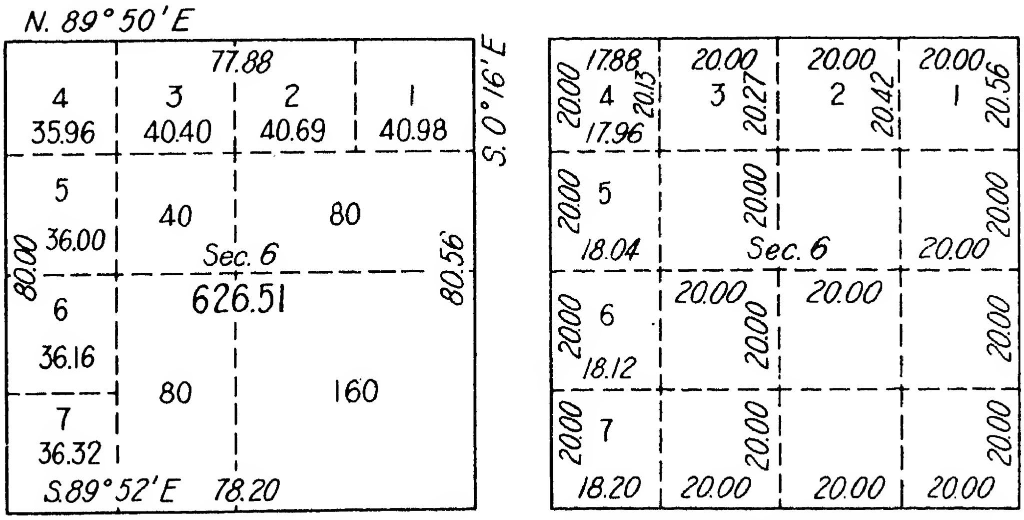


Figure 57. Numbering of sections.



Showing normal subdivision of sections



Showing areas

Showing computed distances

Figure 58. Subdivision of sections.

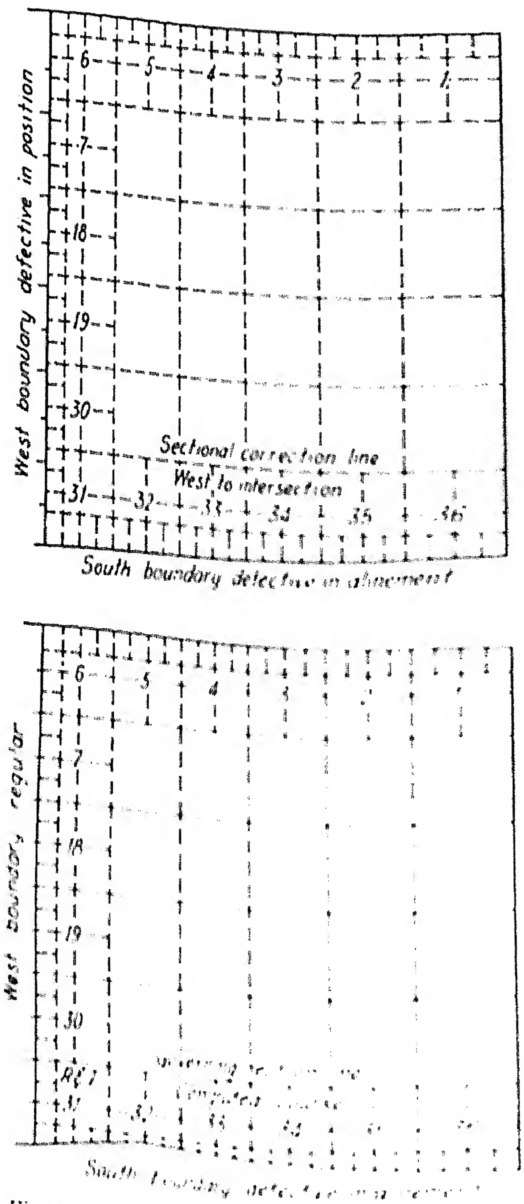


Figure 59. Subdivision of townships with defective boundaries.

a. *Post and Tree Monuments.* The usual wrought-iron post monument carries a brass cap. Identifying capital letters and Arabic numerals are stamped on the cap with steel dies. Similar letters and numbers are marked on tree monuments, on the trunk just above the root crown, using a timber scribe. If the tree is smooth barked, the markings are cut directly into the bark; if it is rough barked, a long blaze is made on the trunk to receive the markings. The following schedule shows the ordinary markings common to all classes of corners and accessories.

Marks	*To indicate*	*Marks*	*To indicate*
A M	Amended monument.	R	Range.
A M C	Auxiliary meander corner.	R M	Reference monument.
A P	Angle point.	S	Section.
B O	Bearing object.	S	South.
B T	Bearing tree.	S C	Standard corner.
C	Center.	SE	Southeast.
C C	Closing corner.	S M C	Special meander corner.
E	East.		
L M	Location monument.	SW	Southwest.
M	Mile.	T	Township.
M C	Meander corner.	TR	Tract.
N	North.	W	West.
NE	Northeast.	W C	Witness corner.
NW	Northwest.	W P	Witness point.
P L	Public land (unsurveyed).	¼	Quarter-section.
		1/16	Sixteenth-section.

The markings on the brass cap of an iron post are made to read from the south side of the monument. These markings (fig. 60) include the year during which the monument was set.

b. *Stone Monuments.* Where stone corner monuments are used, the letters, numbers, and other markings are chiseled into one or more of the vertical faces or edges of the stone rather than upon its top. Stone monuments are set with either their faces or their edges in the cardinal directions, depending upon the location of the corner in the survey. Stone township corners bear the letters and numbers indicative of the townships and ranges to which the particular monument is common. If the monument is on a base line or standard parallel, it must also show the letters *S C* or *C C* to distinguish between a standard or closing township corner. Section corners are likewise distinguished as standard or closing corners. In addition, they bear a number of grooves in the appropriate faces or a number of notches in the appropriate edges of the monument. These notches or grooves indicate, for exterior section corners, the number of miles from the monument to the adjoining township corners. For subdivisional corners, they show the number of miles from the monument to the east and south township boundary lines, thus furnishing a means of identifying the sections which meet at the monument.

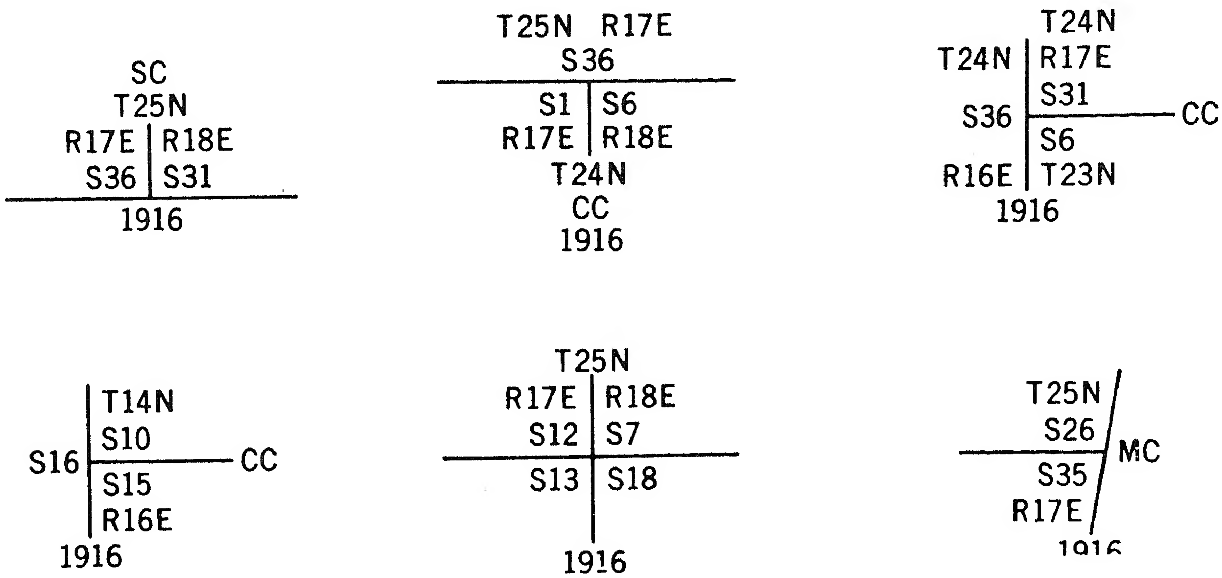


Figure 60. Typical identifying letters and numerals on monuments.

146. Subdividing Sections and Fractional Sections

The subdivision of sections and fractional sections is not a normal function of the Bureau of Land Management. Such surveys are customarily executed by local surveyors. Before a section is subdivided it is necessary to identify the section and quarter-section corners or to relocate them.

a. Subdivision into Quarter-Sections. When the quarter-section corners have been identified, two straight lines are run between opposite quarter-section corners to divide the section into its four quarters. The intersection of these lines is monumented and is the legal center of the section. When opposite quarter-section corners have not been or cannot be fixed, as when one falls in a lake or river or falls outside of the public-land boundaries, subdivision-of-section lines are run, from those quarter-section corners which have been located, parallel to the north, south, east, or west lines of the section, as the case may be, to the water course, reservation line, or other boundary of the fractional section, in accordance with the lines indicated upon the official plat.

b. Subdivision of Quarter-Sections. In subdividing quarter-sections, the quarter-quarter- or sixteenth-section corners are established at points midway between the section and quarter-section corners, and between the quarter-section corners and the center of the sections. An exception to this is made on the last half-mile of those lines closing on township boundaries. Here the corners are set at 20 chains, proportionate measurement, from the preceding quarter-section corner so that survey discrepancies are placed in the last distance. When the sixteenth-section corners have been established, straight lines are run between opposite corners to divide the quarter-section into sixteenths of a section. Subdivisional lines of fractional quarter-sections are run from established sixteenth-section corners, with courses controlled by the conditions represented upon the official plat, to the waterway, reservation, or other irregular boundary which causes such subdivisions to be fractional. In a manner similar to the procedure outlined above, sections can be subdivided into parts which are, respectively, $\frac{1}{64}$th, $\frac{1}{256}$th, and $\frac{1}{1024}$th section.

147. Fractional Lots

The provisions of paragraph 146 apply to sections of normal size and to fractional parts thereof. Those sections bordering on the north and west boundaries of a township, excepting section 6, are subdivided by protraction on the official plat into parts representing two half-

quarter sections and four lots, the latter containing the fractional areas resulting from the plan of subdivision of the townships and from accumulated discrepancies in the survey (fig. 58). The lots are numbered from 1 to 4 in an east to west direction or in a north to south direction except in section 6 which lies in the northwest corner of the township. Here there are seven fractional lots, numbered as indicated in figure 58. Similar fractional lots may be formed on the south and east sides of a township when these boundaries are defective (fig. 59), and in interior sections (para. 144). Fractional lots are surveyed in the field in accordance with the data shown on the official plat of the township, adopting proportionate measurements between controlling corners.

148. Townsites

Townsites in the public lands are those areas within one or more townships which are divided into streets, alleys, and blocks of lots. Many townsites, particularly those developed on the relatively flat prairies and plains, follow along rectangular lines closely correlated with the regular lines of the survey system. Other irregular townsites came into being, prior to subdivision surveys in the area, around trading posts and military establishments. The townsite boundaries are surveyed, closed, and tied in with the lines of the public lands system. Monuments are placed at the intersections of all such lines with the townsite boundaries. All street, block, and lot lines are likewise staked out and a plat of the area prepared, showing the relationship of public survey lines to all lines in the townsite.

149. Location of Islands

The beds of navigable bodies of water below the high water line do not form a part of the public domain. The sovereignty to such lands lies in the individual states in which the waterways are located. However, in the case of islands which were above high water prior to the date of admission into the Union of the state in which the islands are located, title was vested in the United States. Such islands are part of the public domain and are subject to survey. The islands are therefore to be located by triangulation, direct measurement, or other suitable method, their shore lines located by a meander traverse, and the islands shown on the official plat or plats. Township or section lines which traverse the island are located and regular township, section, and meander corners are set. When the size of the island warrants, subdivision-of-section lines are protracted normally thereon.

Section III. FIELD OPERATIONS IN SURVEYS OF PUBLIC LANDS

150. Establishing Parallels of Latitude

The base line, standard parallels, and latitudinal township lines are intended to be true parallels of latitude. These parallels are perpendicular, at any point, to the direction of the meridian passing through that point. Since the meridians converge, the parallel is a curved line. The departure of the curve from a straight line is not great in a short distance and two points, 20 chains apart, on a given parallel may be said to define the direction of the curve at either point without appreciable error.

151. Meridians

The east boundary of a quadrangle or township is the principal meridian. Section-line meridians are parallel to the east boundary meridian. The convergence of meridians will cause the northern boundary to be shorter than the southern boundary and will also result in the section-line meridians having a slight bearing to the left. The amount of this convergence must be taken into account in computing the closure of the tract or township. The Bureau of Land Management *Standard Field Tables* gives corrections for convergence within a township.

152. Normal Procedure in Running Section Lines

Figure 61 shows the normal order of running lines in subdividing a township. Starting at the southwest corner of section 36, line 1 is run northerly parallel to the east boundary of the township. Quarter-section and section corners are set along this line at distances of 40 and

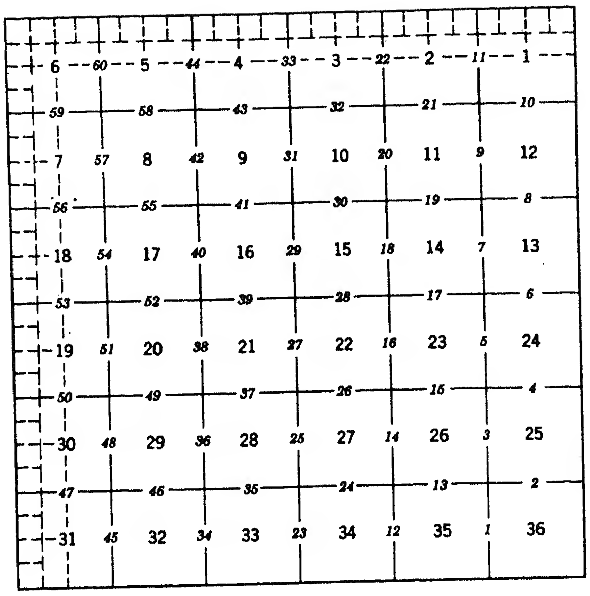


Figure 61. Normal order of running section lines.

80 chains respectively. From the section corner just established, a random line, 2, is run easterly parallel to the south boundary of the township, a temporary quarter-section corner being set on this line. The "falling" of this line is determined. This is the distance from the point where the random line intersects the guide meridian to the standard corner previously set on the guide meridian. If the "falling" of the line is within allowable limits, the direction of the return course which will join the section corners is computed. This return line is then run, a permanent quarter-section corner being set midway along it. This process is continued for section 25 by running line 3 northerly and parallel to the east township boundary and then running the random line 4 easterly and closing back as before. Successive sections in the range are monumented until section 1 is reached. As indicated by the numbered lines in figure 61, the process is repeated for each range in the township, the layout of the sections in the second range starting with line 12. On the last range, the random line 46 is run to the east and the random line 47 is run to the west.

153. Marking Meander Corners and Meander Lines

a. Meander Corners. All navigable bodies of water and other important rivers and lakes are segregated from the public lands at mean high water level. When a survey line intersects such a body of water a *meander corner* is established. The distance from the nearest section or quarter-section monument on the survey line is measured and recorded. The monument is marked *M C* on the side facing the water. Other markings (fig. 60) refer to township, sections, and date of establishment. Such a monument is referenced by bearing trees or other convenient accessories. Where the width of the water course permits, the survey line is carried across to a meander corner on the far shore, the distance between monuments is obtained by triangulation or direct measurement, and the survey is continued.

b. Meander Lines. The traverse of the margin of a permanent natural body of water is called a *meander line.* Meander lines are not run as boundaries of the waterfront property. Meander lines are run between fixed monuments to permit computation of a closed traverse.

154. Defining Riparian Rights

Owners of waterfront property hold title to the *ripa* or bank, such ownership extending, in general, to the mean high water line. When by action of water, the bed of the body of water slowly changes its position, the high water mark also shifts and ownership of adjacent land progresses with it. Riparian owners have numerous other rights, including that of access to navigable water which involves the right to build a wharf or pier out from shore. Such a wharf must not extend beyond the established pierhead line or it would interfere with navigation. Surveys are therefore required to reestablish such lines.

155. Apportioning Excess and Deficiency

It has already been indicated that the deficiency in dimensions and area which results from the convergence of the meridians is placed normally in the fractional lots bordering on the westerly boundary of a township. Similarly, the excess or deficiency which results from discrepancies between the meridional measurements on the exterior boundaries and on the interior subdivision lines is placed in the fractional lots bordering on the north boundary of

the township. When a regular section or quarter-section is to be subdivided into a number of aliquot parts and an excess or deficiency is found in the distance between established monuments, this excess or deficiency is apportioned equally among the parts.

156. Identifying Tracts

Before a tract or other subdivision can be resurveyed or subdivided, it must be identified and its bounding monuments recovered. Paragraph 145 contains information on the identifying markings on monuments.

157. Identification of Existent Corners

In making resurveys or subdivision surveys in areas where the original public lands surveys were carried out many years ago, considerable difficulty may be encountered in recovering and identifying existent corners. The monuments which were used include marked trees, deposits of durable material, chisel-cut rocks, marked wooden posts or stakes, marked stone or concrete bounds, brass tablets set in ledge, and the standard wrought-iron post. Accessories, to assist in the location and identification of monuments, include bearing trees, bearing objects, mounds, and pits. Over the years many witness, line, and corner trees have been cut down, mounds have been leveled and pits filled, and the less durable monuments have been obscured, obliterated, or destroyed. To provide data for use in his research, the surveyor should make application to the appropriate office for permission to make or to obtain copies of the original plats and field notes. For a list of these offices, reference should be made to the Manual of Surveying Instructions of the Bureau of Land Management. The surveyor should also obtain all possible information regarding subsequent surveys and the names of adjacent property owners for the tract in question. From examination of the notes and plats of original and subsequent surveys, by physical examination in the field, and from testimony of residents in the area, it will be possible to find positive evidence of many obscured or obliterated corners. A few monuments may be totally destroyed or lost and will have to be restored.

158. Restoration of Lost Corners

A lost corner is a point of a survey the position of which cannot be determined, beyond reasonable doubt, either from traces of the original marks or from acceptable evidence or testimony which bears upon the original position, and the location of which can be restored only by reference to one or more interdependent corners. In the search for a corner, retracements will have been made from located monuments on all lines which meet at the corner so that the total resurvey distances between located monuments on either side of the lost corner will be known and can be compared with distances shown in the original notes. This comparison normally will permit the restoration of the corner by application of the principle of proportionate measurements. The methods used in restoring such lost corners are given in the following paragraphs.

159. Double Proportionate Measurement

The method of double proportionate measurement is generally applicable to the restoration of a corner which is common to four townships or which is common to four sections. A proportionate measurement is one that gives a concordant relationship between all parts of a line. That is, the new values given to the several parts, as the result of remeasurement, will bear the same relation to the lengths of the same parts in the record of the original survey as the new measurement of the total length bears to the total length shown in the record. The term double proportionate measurement is applied to measurements made between two known corners on a latitudinal line and between two known corners on a meridional line for the purpose of reestablishing a lost monument at the intersection. The effect of double proportionate measurement is that the distances will control the relocation rather than the direction of the lines as indicated in the original record. A case in point (fig. 62) shows a lost corner X which is common to four townships. Points A and B represent recovered corners on the range line and points C and D represent recovered corners on the latitudinal township line. A retracement is first made between the nearest known corners (A and B) on the meridional line, north and south of the missing corner, and

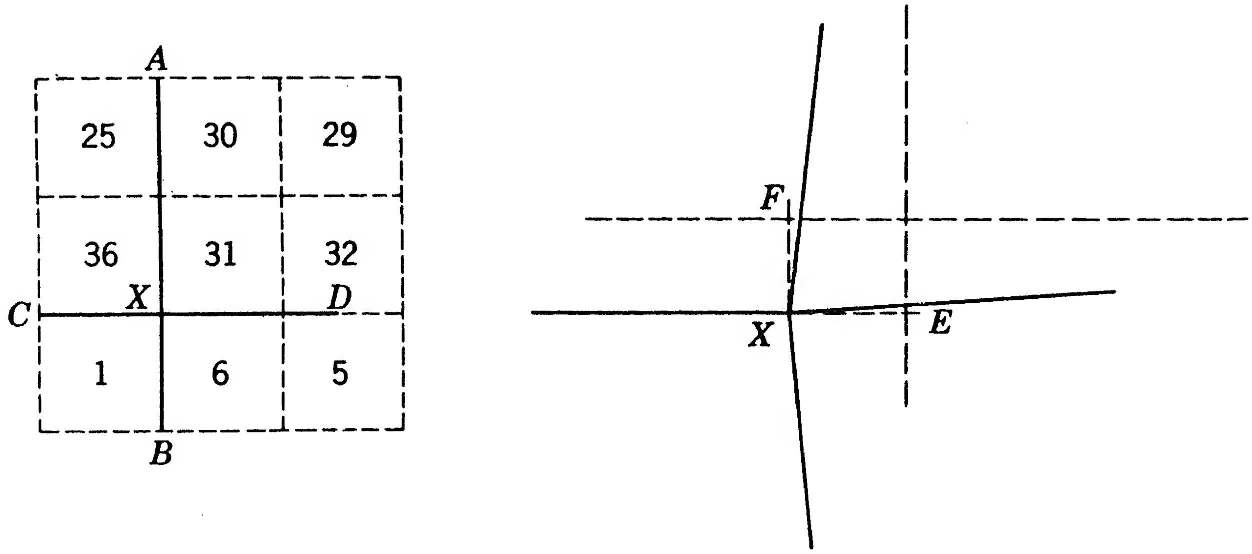


Figure 62. Restoration of lost corner by double proportionate measurement.

upon that line a temporary stake E (fig. 62) is placed at the proper proportional distance. This point E determines the latitude of the lost corner. In setting point E, the distance BE is to the original record distance BX as the total retracement distance BA is to the distance BA as shown in the record. Next the nearest corners (C and D) on the latitudinal line are connected and a temporary stake F is placed on this line at the proper proportional distance. This point F determines the departure or longitude of the lost corner. Then a line is run, east or west, as the case may be, from the temporary stake E on the meridional line and a line is run north or south, as relative situations may determine, from the temporary point F on the latitudinal line. The intersection of these two lines at X fixes the position at which a monument is to be set to restore the lost corners.

160. Single Proportionate Measurement

The method of single proportionate measurement is generally applicable to the restoration of lost corners on standard parallels and other lines established with reference to definite alinement in one direction only. Intermediate corners on township exteriors and other controlling boundaries are also restored by this method. A retracement is made connecting the nearest identified regular corners on the line in question and on opposite sides of the missing corner. In running the retracement line, a temporary stake is set on the trial line at the original record distance for the missing corner. The total distance between recovered monuments is measured and the "falling" (para. 152) at the objective corner is also determined. On meridional township lines an adjustment is made at the temporary stake for the proportional distance along the line. Then the stake is set over to the east or to the west a distance which bears the same relationship to the total "falling" at the objective corner as the distance of the missing corner from the point of beginning bears to the total length of the retracement line. On standard parallels and on latitudinal township lines three adjustments are necessary. The temporary stake is first set forward or back the proportional part of the difference between the record distance and the retracement measurement; it is then set over to place it on the true latitudinal curve; and lastly, a correction is made for the proportional part of the "falling" at the objective corner. The adjusted position is thus placed on the true line which connects the nearest identified corners, and at the same proportionate distance from each as existed in the original survey. Any number of intermediate lost corners may be

restored during a single retracement by setting a temporary stake for each during the retracement survey and making appropriate adjustments to the position of each such stake. Lost meander corners, originally established on a line projected across the meanderable body of water and monumented on both shores, are restored by single proportionate measurement, after the section or quarter-section corners upon opposite sides of the missing meander corner have been identified. Where these adjacent corners have been obliterated as well but where there is evidence of the stability of the shore and no indication of erosion or accretion, the shore itself may be taken as an identified natural feature and the lost corner replaced with reference to the shore.

161. Restoration of Closing Corners

A lost closing corner is reestablished on the true line that was originally closed upon, and at the correct proportionate distance between the nearest regular corners to the right and left. In restoring a lost closing corner on a standard parallel or other controlling line, this controlling line is retraced, beginning at the corner from which the line was originally run. A temporary stake is set at the record distance for the closing corner, and the total distance and "falling" determined at the next recovered regular corner beyond the missing monument. The temporary stake is then adjusted in position as in single proportionate measurement.

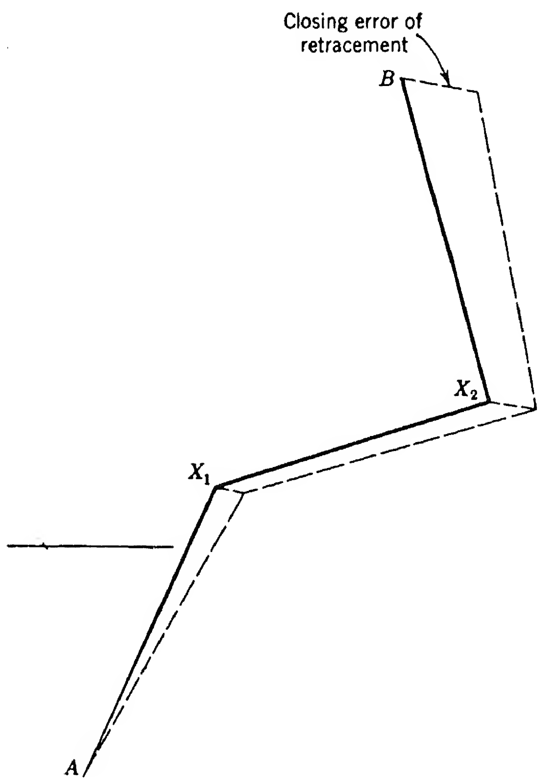


Figure 63. Retracement of broken boundary.

162. Restoration of Broken Boundaries

In certain instances it is necessary to restore the angle points, within a section, of the meander courses for a stream or other body of water. The positions of the meander corners on the section boundaries are first recovered or restored. The record meander courses and distances are then run, setting temporary angle points. If the end of the last course run fails to fall on the objective meander corner (fig. 63), the distance and bearing from the end of the last course to the closing meander corner is measured and recorded. Each temporary angle stake is then moved in a direction to reduce the closing error along a line having the same bearing as the closing error. The distance which each such stake is to be moved bears the same relationship to the total closing error as the distance of that stake from the point of beginning (measured along the meander courses) bears to the sum of the lengths of all the courses.

163. Restoration by Original Control

Where a line has been terminated by measurement in one direction only, a lost corner is restored by running a line from the nearest identified regular corner on the original survey line, using the record distance and bearing to reestablish the missing monument. Such corners are encountered where lines were discontinued at the shore of a wide meanderable body of water or at the edge of ground classed as impassable.

164. Index Correction for Average Error in Alinement and Measurement

In cases where a retracement has been made of many miles of the original lines, between identified original corners, and there has been developed a definite surplus or deficiency in

measurement, or a definite angle from cardinal which characterized the original survey, it is proper to make allowance for the average differences. An adjustment is taken care of automatically in all cases where there exists a suitable basis for proportionate measurement, but where such control is lacking in one direction, an average difference, if conclusive, is used by applying this difference to the record courses and distances.

165. Special Cases of Corner Restoration

Much experience, painstaking attention to detail and to every bit of valid corroborative evidence, and sound judgment are essential if the surveyor is to correctly retrace the boundaries of a tract on which many of the corner monuments have become obscured, obliterated, or lost, and when accessories have been destroyed. Through accumulated years of experience with situations of all types, the cadastral engineers of the Bureau of Land Management have developed competence with respect to resurveys of public lands beyond that which the military surveyor can reasonably expect to acquire. The records of the regional offices, of the public survey offices, and of the Washington office form a vast reservoir of experience in dealing with special and unusual cases. The plats; field notes; reports of field examinations; office opinions; and decisions of the bureau, of the Attorney General, and of the courts are available and are freely drawn upon when needed to assist the surveyor in situations which are beyond his experience or which are not specifically covered in the Manual of Surveying Instructions. When faced with such a situation, the surveyor must report all facts which he is able to gather to the appropriate administrative office.

166. Plats of Surveyed Lands

The official plat of a township or other subdivision is the drawing on which is shown the direction and length of each line surveyed, established, retraced, or resurveyed; the relationship to adjoining official surveys; the boundaries, designation, and area of each parcel of land; and, insofar as practicable, a delineation of the topography of the area and a representation of the culture and works of man within the survey limits. A subdivision of the public lands is not deemed to have been surveyed or identified until the notes of the field survey have been approved, a plat prepared, the survey accepted by the Director of the Bureau of Land Management as evidenced by a certification to that effect on the plat, and the plat has been filed in the district land office. Figure 64 shows a typical township plat. The original drawing shows both a graphical scale and a representative fraction for both the township as a whole and for the enlarged diagram. Because the plat has been photographically reduced, the representative fraction and scale are no longer true. Plats are drawn on sheets of uniform size, 19 x 24 inches in trimmed dimensions, for convenience in filing. The usual scale is 1 inch equals 40 chains, equivalent to a representative fraction of 1:31,680. Where detail drawings of a portion of the survey area are required, scales of 1 inch equals 20 chains or 1 inch equals 10 chains may be used. A detail of a small area may be shown (fig. 64), as an inset on the main plat. Larger details are drawn on separate sheets. When the drawing is simple, with few topographic or hydrographic features or works of man to be shown, the entire drawing is in black ink. When, as in figure 64, the features other than the survey lines are quite extensive, color printing is used. Survey lines, numbers, lettering, and railroads are printed in black; topographic relief, roads, highways, trails, culture, alkali flats, sandy-bottom draws, and sand dunes are shown in brown; rivers, lakes, streams, and marshes are shown by conventional symbols in blue; and timbered areas are indicated in green. Where such a green overprint might obscure other details, the presence of timber may be indicated in a note (fig. 64). These several colors are not shown on the reproduction of the plat presented in figure 64 although the various features are indicated in appropriate colors on the original.

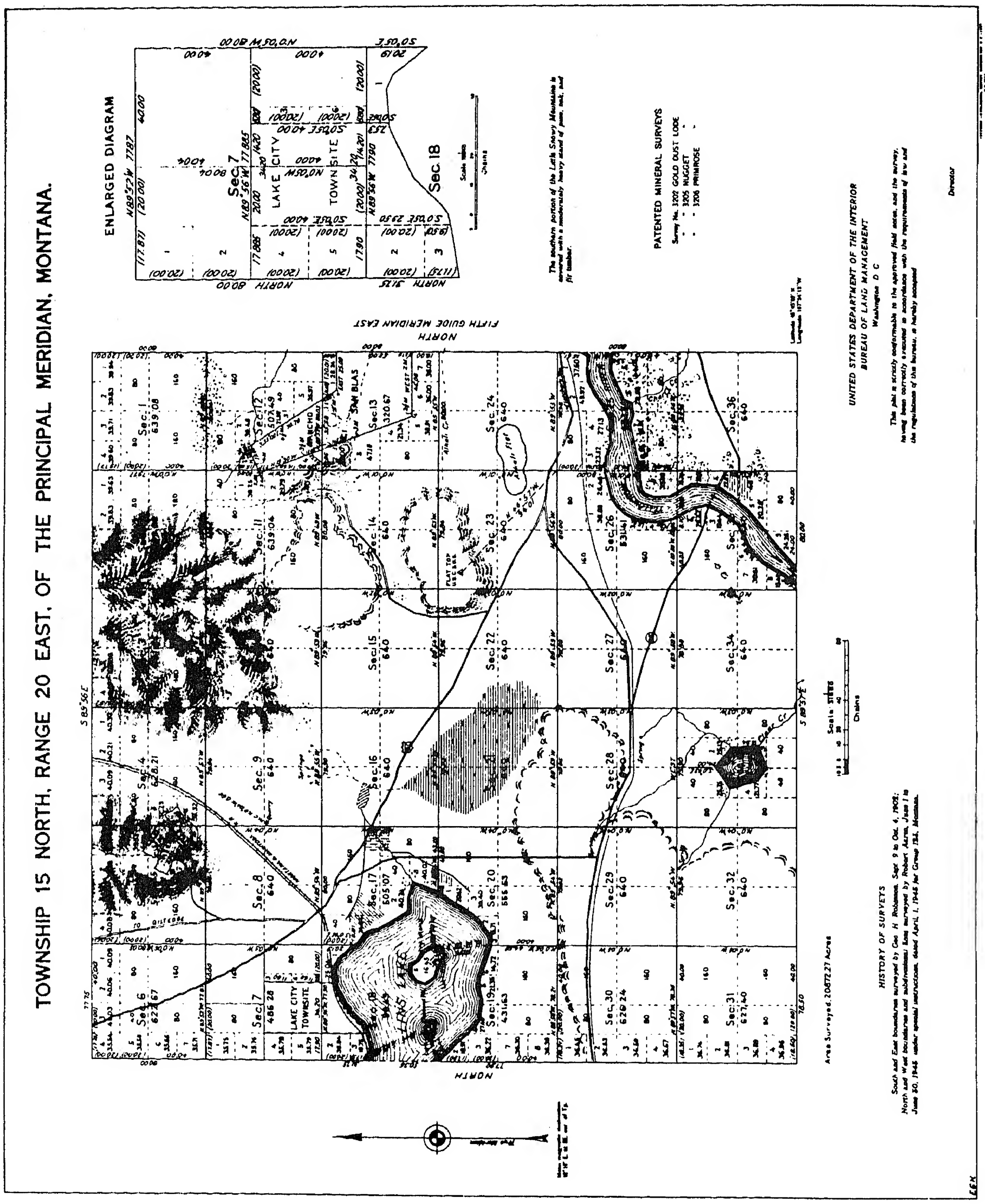


Figure 64. Typical township plat.

167. Functions and Responsibilities

In resurveying property boundaries and in carrying out surveys for the subdivision of land, the surveyor has the following functions, responsibilities, and liabilities:

a. Locate in the public records all deed descriptions and maps pertaining to the property and he must properly interpret the requirements contained therein.

b. Set and properly reference new monuments and replace obliterated monuments.

c. Be liable for damages caused by errors resulting from incompetent professional work.

d. Attempt to follow in the tracks of the original surveyor, relocating the old boundaries and not attempting to correct the original survey.

e. Prepare proper descriptions and maps of the property.

f. May be required to connect a property survey with control monuments so that the grid coordinates of the property corners can be computed.

g. Report all easements, encroachments, or discrepancies discovered during the course of the survey.

h. When original monuments cannot be recovered with certainty from the data contained in the deed description, seek additional evidence. Such evidence must be substantial in character and must not be merely personal opinion.

i. In the absence of conclusive evidence as to the location of a boundary, seek agreement between adjoining owners as to a mutually acceptable location. The surveyor has no judicial functions; he may serve as an arbiter in relocating the boundary according to prevailing circumstances and procedures set forth by local authority.

j. When a boundary dispute is carried to the courts, he may be called upon to appear as an expert witness.

k. Must respect the laws of trespass. The right to enter upon property in conducting public surveys is provided by law in most localities. In a few political subdivisions, recent laws make similar provision with respect to private surveys. Generally, the military surveyor should gain permission from the owner before entry on private property. Lacking permission from an adjoiner, it is usually possible to make the survey without trespassing on the adjoiner's land but such a condition normally adds to the difficulty of the task.

l. Be liable for actual damage to private property resulting from his operations.

168. Use of Deeds and Records in Property Surveys

a. Use of Recorded Deeds. When a military surveyor is required to survey a property, the deed may be furnished to him by higher military authority. Frequently he must look in the public records for the recorded copy of the deed, for filed plans or maps, or for the deeds to adjacent properties.

b. Registry of Deeds. Land records are filed in a registry of deeds. The statutes of various localities differ in their designation of the recording official. He may be a town or city clerk or registrar, a county registrar, or other legally constituted recording officer. Frequently the recording of deeds is a function of the county government and each county will maintain one or more registries of deeds or halls of records serving the whole or a designated part of the area of the county.

c. Grantor and Grantee Indexes. The index of deeds at the registry is in two parts, one listing each deed under the name of the grantor or seller of the land and the other containing the listing under the name of the grantee or purchaser. The index is frequently further divided by years, so that the surveyor should ascertain the name of the buyer or seller of the property and the approximate date of the transfer of the property. He then enters the grantor or grantee index for the proper year and, under the name of the party, finds the date of the transaction, the number of the deed book, and the page of that book on which the deed is recorded.

d. The Deed Book. The appropriate page of the deed book contains a copy of the deed in

question. It may contain a reference to a filed map, particularly when the property was once a lot of a larger tract. Reference numbers in the text or on the margin refer to the next preceding transfer of the property or the assignments or attachments on the property. When the deed description shows the names of adjoiners, the deeds of these adjoiners may also be looked up. Information from such deeds will often supplement the description of the property in question when such description shows missing or conflicting data. From the deed book or books, the surveyor should be able to copy all necessary data for carrying out the survey.

169. Use of Proportionate Measurements in Resurveys

If, in the course of a resurvey, one or more of the corners are found to have been lost and the distances between recovered corners fail to agree with those given in the deed description, the ratio of the measured lengths to the stated lengths is computed and the missing corners are reset by proportionate measurements. The restoration of lost corners by the methods of double and single proportionate measurements are discussed in paragraph 159 and 160. These methods, as applied in the paragraphs indicated to surveys of public lands, are equally applicable to surveys of private property.

170. Military Property Surveys

Such surveys normally are made in connection with the lease or purchase of private property, in whole or in part, for military use. Occasionally an original survey is required where previous surveys have defined the parcel by reference to natural or artificial boundaries without an actual survey of the dimensions of the property having been made.

171. Essential Elements of the Survey

The surveyor will be furnished a statement from higher military authority as to the extent of the property to be acquired. He must know if it embraces the whole of a parcel or only a portion thereof. In the latter case, he must know the manner in which the parcel is to be subdivided. From such information, it may be possible to compute certain of the distances and bearings of the new lines from data contained in the original land description. Other measurements may have to be taken in the field. The surveyor must—

a. Establish a point of beginning and monument and reference this point.

b. Monument all accessible corners.

c. Witness inaccessible corners by monuments on adjoining lines.

d. Measure the lengths and true bearings on all courses.

e. Ascertain the names of adjoiners.

f. Compute the boundary traverse and apply geometrical checks on the closed figure (TM 5–232).

g. Compute the area of the property.

h. Locate all buildings and improvements on the property by offsets or by supplemental surveys from the property lines.

i. Note all easements or encroachments.

j. Make sufficient measurements to control the location and computation of any curved boundaries.

172. Running the Traverse

Where the boundary lines are free of walls, fences, building lines, hedges, trees, or other obstructions, and where adjacent corners are intervisible, the control traverse for the survey is run along the boundaries from corner to corner, all distances and bearings or angles being measured directly. Where this cannot be done, the traverse is run on offset lines or random lines and sufficient measurements are made to permit the computation of the true lengths and bearings of the boundary lines (TM 5–232).

173. Preparing the Plan

From the notes taken in the field, a scale plan is prepared showing all pertinent information obtained in the course of the survey. The plan should be drawn in ink on tracing cloth of convenient size. It should show the direction of the true meridian; any town, county, or other political subdivision lines, street lines; boundary monuments and lines; fences, walls, buildings, and passageways. The scale should be of such size as to clearly show all necessary details. All pertinent dimensions and bearings must be shown, and also the coordinates of the corners

on the local grid if these have been determined. The plan should show the names of adjoiners; the area of the property; and all easements or encroachments. The title must show the location, scale, and date. The plan should bear the signature of the surveyor and, when required by law, his address and seal. Figure 65 shows a typical plan of a lot, to be acquired as a part of a tract for military use.

174. Writing the Description

In preparing the description of a property, the surveyor should bear in mind that the description must clearly identify the location of the property and must give all necessary data from which the boundaries can be reestablished at any future date. The written description contains the greater part of the information shown on the plan. Usually both a description and plan are prepared and, when the property is transferred, are recorded according to the laws of the county concerned. The metes and bounds description of the property shown in figure 65 is given below.

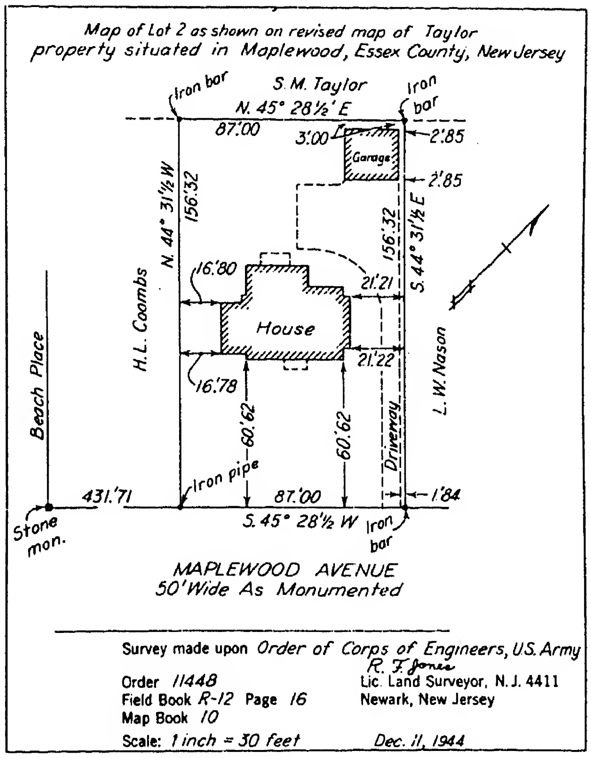


Figure 65. Lot plan.

"All that certain tract or parcel of land and premises, hereinafter particularly described, situate, lying and being in the Township of Maplewood in the County of Essex and State of New Jersey and constituting lot 2 shown on the revised map of the Taylor property in said township as filed in the Essex County Hall of Records on March 18, 1944.

"Beginning at an iron pipe in the northwesterly line of Maplewood Avenue therein distant along same four hundred and thirty-one feet and seventy-one one-hundredths of a foot northeasterly from a stone monument at the northerly corner of Beach Place and Maplewood Avenue; thence running (1) North forty-four degrees thirty-one and one-half minutes West along land of H. L. Coombs one hundred and fifty-six feet and thirty-two one-hundredths of a foot to an iron bar; then turning and running (2) North forty-five degrees twenty-eight and one-half minutes East along land of S. M. Taylor eighty-seven feet to an iron bar; thence turning and running (3) South forty-four degrees and thirty-one and one-half minutes East along land of L. W. Nason one hundred and fifty-six feet and thirty-two one-hundredths of a foot to an iron bar in the northwesterly line of Maplewood Avenue; thence turning and running (4) South forty-five degrees twenty-eight and one-half minutes West along said line of Maplewood Avenue eighty-seven feet to the point and place of beginning; all bearings being true and the lot containing a calculated area of thirteen thousand six hundred square feet. This description has been prepared from a survey made by R. F. Jones, Licensed Land Surveyor, New Jersey No. 4411, said survey being dated December 11, 1944."

CHAPTER 5
HYDROGRAPHIC SURVEYS

Section I. OPERATIONS

175. Use

The principal objective of most hydrographic surveys is to secure information on water areas. The results of the surveys are used for military defense projects, for seaplane anchorages, for sailing directions, for planning harbor improvements, for the studies of erosion, for oceanographic studies and for earth science studies.

176. Types of Surveys

Hydrographic surveying, as treated in this manual, pertains to the measurements performed in making surveys of bodies of water in connection with engineer activities; including instructions for the surveyor concerning tide gaging and depth soundings required for port construction, stream gaging of inland streams, such hydrologic measurements as may be required in water supply and other studies directed by an engineer hydrology team. Such surveys are made for—

a. The location of shorelines.

b. The determination of the depths of waters.

c. The location of shoals and reefs.

d. The determination of the character of the bottom of a body of water.

e. The determination of tidal ranges and types and the elevations of bench marks with respect to tidal datums.

f. The measurement of quantity and velocity of streamflow.

g. The determination of quantities of dredged material.

h. Hydrologic studies concerned with flood-control and water supply.

177. Functions of Charting Agencies

a. Function of U.S. Coast and Geodetic Survey. One of the primary functions of the U.S. Coast and Geodetic Survey is to prepare and publish nautical charts of the coastal waters of the United States and its territorial possessions. The U.S. Coast and Geodetic Survey makes the original hydrographic and topographic surveys to obtain the data for publishing these charts. These charts show the shoreline and the topography adjacent to it; soundings and depth curves; characteristics of the bottom; the location of port facilities, lights, buoys, and other navigational aids; channels, shoals, reefs, and dangers to navigation; the direction of the true meridian and the magnetic variation; and other data of value to navigators. Various scales are employed depending upon the area included on a particular chart.

b. Function of U.S. Naval Hydrographic Office. It is a function of the Hydrographic Office, of the Department of the Navy, to obtain data for and to publish nautical charts of coasts and waters outside the limits of the United States and its possessions for the use of our merchant marine and naval forces. Many data are taken from foreign charts. Other information is obtained from hydrographic surveys carried out from naval vessels.

c. Function of U.S. Lake Survey. The function of the U.S. Lake Survey, operated by the Corps of Engineers, is the preparation and publication of nautical charts covering the Great Lakes system, Lake Champlain, New York canals, and the Minnesota-Ontario border lakes, and the study of all matters affecting the hydraulics and hydrology of the Great Lakes, including the necessary hydrographic and related surveys, investigations, and observations. The work of this office is similar to that of the U.S. Coast and Geodetic Survey in coastal waters.

178. Engineer Applications

Military organizations use the charts produced by the agencies listed in the preceding paragraph whenever and wherever applicable. In addition, military engineers perform surveys concerned with tide gaging, measurement of stream flow, and subaqueous topography as required by military operations. Information in this chapter pertains to methods, techniques, personnel, and equipment for the execution of hydrographic surveys for military usage.

179. Project Planning

a. Use of Data from Prior Surveys. Every effort should be made to obtain and utilize data from previous surveys in the area. This is particularly true with respect to horizontal and vertical control. Soundings and other hydrographic surveys must be made in harbor waters at frequent intervals and charts corrected as channels are widened or deepened, as lights and buoys are installed or altered in position, and as other conditions in the harbor undergo change. (One report of the U.S. Coast and Geodetic Survey indicates that corrections made in a single year to the chart of New York harbor were obtained from 253 different surveys and maps prepared by the Coast and Geodetic Survey, the Corps of Engineers, and the City of New York.) It is unnecessary and uneconomical to reestablish complete horizontal and vertical control for each separate survey. The records of all agencies which have surveyed in the area should be consulted to obtain the positions of triangulation and traverse stations and the elevations of bench marks. Church spires, flagpoles, chimney stacks, lights, radio masts, buoys, and other prominent objects, the locations of which are shown on existing maps or charts, may be used to locate the position of a sounding boat offshore by means of sextant angles. Where such data are taken from foreign charts, the datum and projection used should be checked and the reliability of the chart investigated.

b. Aerial-Photograph Studies. Reference to available aerial photograph of the area will aid greatly in planning the survey. From them a strong and rapidly executed scheme for establishing triangulation or traverse control is often apparent. The outline of shoal-water areas and dangerous submerged reefs is frequently discernible. A number of possible sites which appear to meet the criteria for satisfactory stream-gaging stations may be selected from the aerial photographs, thus minimizing the time required for reconnaissance on the ground. Aerial photographs, corrected for scale and tilt, may be used to prepare a planimetric map of the shore line and to show the location of prominent objects which may be used as signals or ranges in locating soundings.

c. Prevailing Weather Conditions. Data should be gathered with reference to the weather conditions likely to prevail at the time the survey is made, since these will affect both the methods and equipment used. Gales and high seas require sturdier sounding vessels than are necessary in calm waters. Prevelance of fog may necessitate the use of electronics methods (including shoran, electronic position indicator and radar) rather than visual methods of locating soundings. Very cold weather may require that lake or river soundings be taken through holes cut in the ice. Flood flows and floating drift in rivers render the usual stream-gaging methods unsuitable. Insect repellents are required in many areas in warm weather. Cold and high winds necessitate the issue of cold-weather clothing to survey-party personnel. Climatic records and weather data are supplied by the U.S. Weather Bureau and by the following military organizations:

(1) *Air Force.* The Air Weather Service provides—
  - (*a*) Short-term weather predictions available in the field through air weather units at various levels. There is an air weather unit attached to each corps G2 section. There is an air weather center operating in conjunction with JOC (Joint Operations Center) at army level (FM 31–35).
  - (*b*) Long term weather forecasts based on historical records, special studies, and information compiled by the National Intelligence Service (NIS).

(2) *Navy.* The Oceanographic Office, Department of the Navy, prepares short-term and long-term forecasts of sea

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e. The determination of tidal ranges and types and the elevations of bench marks with respect to tidal datums.

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conditions; information is available through liaison personnel.

(3) *Army.*

(*a*) The Army Map Service prepares climatic studies pertaining to terrain conditions, precipitation, temperature, and studies by NIS; available through staff engineer.

(*b*) The U.S. Army Natick Laboratories prepares studies in the field of applied environmental research and can furnish, upon request, up-to-date environmental information as it applies to many practical problems. Appropriate clothing for specific seasons in all world areas is prescribed in TA 50–901 and TA 50–902.

180. Responsibilities Related to Hydrographic Surveys

Responsibilities common to all surveys are covered in paragraph 3. Additional details particularly applicable to hydrographic surveys are itemized below.

a. Investigate requirements for specialized instruments and equipment on the survey. Engineer troops normally will utilize TOE equipment to the extent that such equipment is suitable for the particular hydrographic survey. For extensive surveys, it will be necessary for the engineer staff to consider the need for and to requisition equipment normally not available to survey troops. Specialized instruments include: automatic tide gages and water stage recorders, current meters, sextants, stop watches, sounding lines or poles, and numerous other items described in this manual. Special equipment will include: powerboats, rowboats, buoys, floats, wading boots, and materials for the construction of gage houses, various types of gages, weirs, wire drags, signals, and ranges.

b. Provide for liaison with other government services for the assignment of equipment and operating personnel for certain highly specialized operations. Such arrangements might involve the temporary assignment of a large vessel of the U.S. Coast and Geodetic Survey with all its specialized equipment and highly trained personnel for subaqueous surveys off shore when required in the conduct of military operations.

c. Obtain record rolls and sheets for automatic tide gages and waterstage recorders.

d. Locate secure anchorages for sounding vessels and appropriate camp sites or billets for field personnel.

e. Obtain records and data from other agencies which have surveyed in the area.

f. Where radio instruments are used, comply with all regulations concerning radio procedures and clearances.

g. Where subaqueous surveys are conducted in harbor waters or offshore, comply with established port procedures and obtain any necessary port clearances from the harbormaster.

h. Issue instructions for the use of geographic nomenclature on maps and charts.

i. Ascertain that complete instructions governing the conduct of the work are issued to all field parties and are fully understood. In sounding operations in particular, control in the field is far more difficult to maintain than in other types of surveying. A very high degree of coordination is required between boat and shore parties. The use of portable radio telephones, when available, will assist greatly in coordinating these operations. Watches carried by shore and boat parties and by the tide-gage observer must be synchronized.

Section II. TIDAL OBSERVATIONS

181. The Tide and Tide-Producing Bodies

The rhythmic rise and fall of the surface of the ocean known as the *tide* results from the gravitational attraction of the moon and the sun acting upon the rotating earth. This vertical movement of the surface of the sea is accompanied by horizontal movements, particularly manifest in narrow straits and estuaries, known as *tidal currents.* When the water is rising and moving inshore it is the *flood* tide; when it is falling and moving seaward it is the *ebb* tide. The maximum height to which

the sea rises is called *high water;* the minimum level is *low water.* The difference between the two is the *range* of the tide. The moon, because of its relative closeness to the earth, is the chief tide-producing body. Although the sun is far larger, its distance is so great that its tide-producing effect is only about two-fifths that of the moon. For the planets and other celestial bodies, the combined effects of size and great distance from the earth are such that they have no measurable influence on the ocean tides.

182. The Cause of the Tides

The principal cause of the tides is the difference in gravitational attraction exerted by the moon upon different parts of the earth. The secondary cause is the similar difference in the attraction of the sun. The attraction varies as the square of the distance between the earth and the moon (or sun), so that the portion of the earth's surface nearest the moon is attracted more powerfully than the central portion. The latter, in turn, is attracted with a greater force than that portion of the earth's surface which is most remote from the moon. Assuming that the surface of the earth to be covered by water, that these waters would instantly respond to the lunar tide-producing force, that the moon is in the plane of the earth's surface, and disregarding friction and inertia, the conditions indicated in figure 66 would exist. The highest tides, at any instant, would be on the equator directly under the moon (at A), and at a point on the opposite side of the earth (B), with the lowest tides in a belt extending around the earth on either side of a plane passing through the poles. The tidal range would vary from a maximum at the equator to zero at the poles. An observer on the rotating earth at C, would note two high waters of substantially equal height and two low waters, again of equal height, during the interval between two successive transits of the moon over this meridian. This interval is known as the *lunar day* and has a mean value of 24.84 clock hours. Thus, at C, where the tide is said to be typically *semidiurnal* in character, there are two high and two low waters in a period of about $24^h\ 50^m$. The interval between successive high waters (or between successive lows) will be about $12^h\ 25^m$. The interval between high tide and the succeeding low will be approximately $6^h\ 13^m$. This highly simplified explanation of the lunar tide must be modified to account for the changing positions of the moon and sun, the effects of water depths and the continental shores, and other factors. The major effects are covered in the paragraphs 183–188.

183. Effects of Change in the Moon's Declination

The declination of the moon (its angular distance north or south of the plane of the earth's equator) will vary over a period of time. This results from the fact that the plane of the moon's orbit of revolution around the earth is inclined to the plane of the equator. Figure 67 indicates, for the same basic assumptions made in the preceding paragraph, the tidal conditions when the moon has a plus (north) declination. At the equator the two high waters are still equal and the tide is semidiurnal in character. For an observer at B, with the moon on the meridian, high tide would be greater than the average. Twelve lunar hours (12.42 solar hours) later, when the observer is at B', the high water will be less than the average. There will thus be a *diurnal inequality* between the heights of the two tides. An observer near the poles would note but one high and one low water during the lunar day. The tide would have *diurnal* characteristics at this point.

184. Effect of Relative Positions of Moon and Sun

The period of revolution of the moon around the earth, with respect to the plane passing

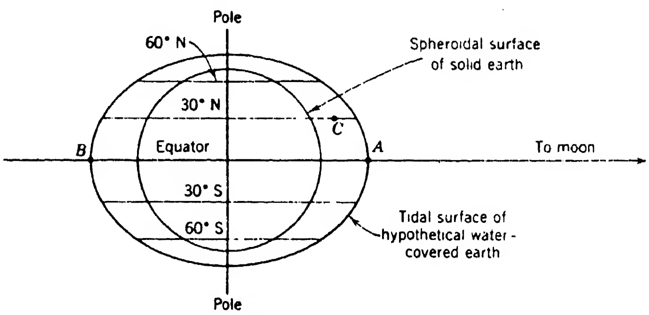


Figure 66. Lunar tide with moon's declination 0°

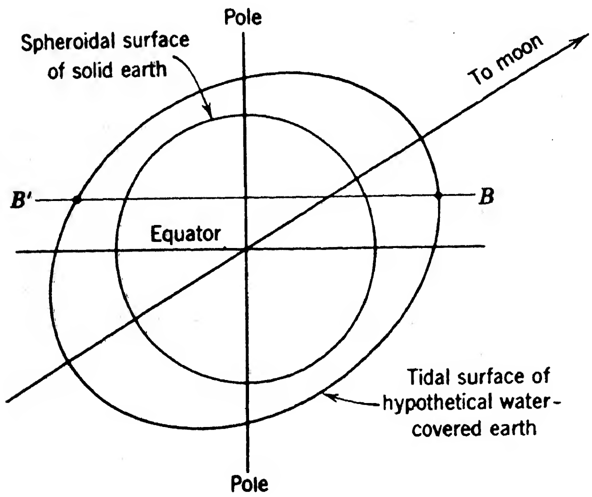


Figure 67. Lunar tide when moon has maximum positive declination.

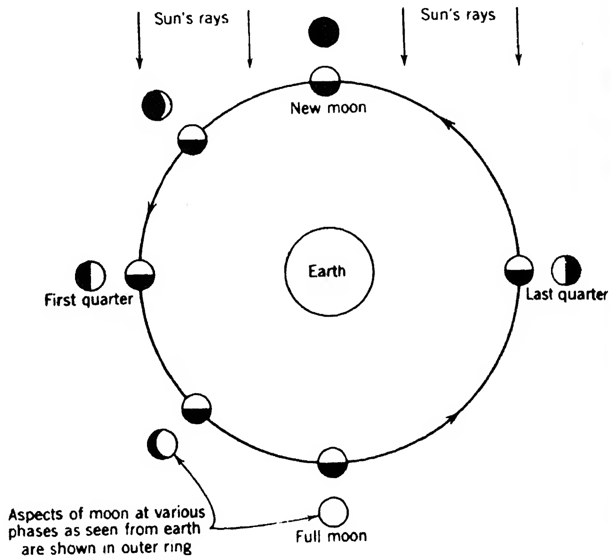


Figure 68. The phases of the moon.

through the earth and the sun, is known as the *lunar* (or *synodical*) month. The period is approximately 29½ days. The relative positions of the moon, with respect to the sun and earth, during this period are known as the phases of the moon (fig. 68). When the moon and the sun are acting in the same plane, at new moon or full moon, their tidal effects are additive, producing tides that are higher than usual. These are called *spring* tides. When the moon is at *quadrature* (*first* or *last quarter*) the tides produced by the sun and moon partially neutralize each other and the tidal range is less than normal. These tides are called *neap* tides.

185. Effect of Variable Distance of Moon and Sun

The eccentricity of the moon's orbit is large, the actual distance from the earth to the moon varying by about 13 percent. As a result, the tide-raising force varies throughout the lunar month. The tidal range is some 20 percent greater when the moon is nearest the earth, at *perigee,* than is the case when the moon is farthest from the earth, at *apogee.* The eccentricity of the earth's orbit about the sun is less, the variation in distance being only about 3 percent, but the solar tide will be somewhat greater at *perihelion,* when the earth is closest to the sun about 1 January, than at *aphelion,* about 1 July of each year.

186. Effect of Longitude of Moon's Node

The effect upon the range of tides which is known as that of the longitude of the moon's node results from cyclical variations in the declination of the moon. The duration of this cycle, that is, the period required for the longitude of the moon's node to pass through a complete range from 0° to 360° is little less than 19 years. Tidal observations continued over a 19-year period will include the effects of all variations in the moon's declination. Mean tidal ranges and the elevations of tidal datums determined from a shorter record must be corrected for the effect of the longitude of the moon's node (para. 212). For observations of less than a year, a correction must also be made for the effect of variations in the declination of the sun.

187. Effect of Wind and Barometric Pressure

The height and time of occurrence of high or low water may vary considerably from normal at any place because of changing weather conditions. If the wind blows steadily from the sea into a bay, the water is piled up and the height of the tide is increased. The time of high water is delayed because the water continues to flow in after the computed time of high water has passed. The maximum water level is not observed until the effect of

the ebb current balances the effect of the wind. The reverse of this situation will occur when the wind is blowing out of an estuary. When the barometric pressure is above normal the water surface will be depressed. It will be elevated when the pressure is low. The water level will change about 1 foot for a pressure change of 1 inch on a mercury barometer. In most instances the effects of weather conditions are irregular and unpredictable. They tend to balance out in a long series of observations. In some localities, weather effects occur with sufficient regularity to be allowed for, partially, in tide predictions.

188. Effects of Water Depths and Shoreline Configuration

Shallow waters and narrow inlets will greatly affect tidal ranges and the times of occurrence of high and low water. These values may be quite different at points relatively close together.

189. Lunitidal Interval

Because of inertia, friction, coastal configuration, water depth, and other factors, there is a lag between the time of transit of the moon over a meridian and the time of the next high water at that place. This interval, which varies somewhat throughout the lunar month, is known as the *lunitidal interval.* Determined on the days of new and full moon, it is known as the *vulgar or common establishment of the port.* When observations of the lunitidal interval have been made at a place for at least a lunar month, a mean value is taken to give the *corrected establishment of the port or the mean high water lunitidal interval.* Details of the determination of lunitidal interval are given in paragraph 210. Once this value becomes known it is a simple matter to determine the approximate daily times of high and low water by computing the standard time of the moon's local upper transit and applying the lunitidal interval to this time, remembering that low water will occur about $6^h\ 13^m$ after high water.

190. Priming and Lagging of the Tides

The common establishment of the port is equal to the lunitidal interval on days of new or full moon. For a few days after these dates the crest of the combined tidal wave resulting from the action of the sun and moon lies to the west of the moon's tide and high water occurs earlier, that is, the lunitidal interval is shorter. This is called the *priming* of the tide. Shortly before new or full moon the crest of the combined tide is east of the moon's tide and the lunitidal interval is lengthened. This is known as the *lagging* of the tide.

191. Daily Change in Time of High and Low Water

Since the lunar day, or the period of rotation of the earth with respect to the moon, averages 24.84 solar hours, the moon will cross a given meridian about 50 minutes later each day. The times of high and low water will, on the average, be later each day by this same interval.

192. Types of Tides

The actual tide at any place can be analyzed best as one compounded of a number of partial solar and lunar tides, each having semidiurnal, diurnal, or long-period components, together with partial tides resulting from meteorological and shallow-water factors. In certain localities the semidiurnal constituents are predominant and the tide is characteristically *semidiurnal* in type. At other points the diurnal constituents control and the tide is typically diurnal. In still other parts of the world, the semidiurnal and diurnal constituents combine to produce a tide of *mixed* type. In the latter case there is a marked diurnal inequality both in the heights of consecutive high or low waters and in the time intervals occurring between them. Typical traces (fig. 69) of the three general types of tide (semidiurnal, diurnal, and mixed) are plotted with ordinates in feet and abscissas in hours. Figure 70 shows a characteristic semidiurnal tide for a single month at a port of embarkation. The diagram was constructed by plotting times and heights and joining them by straight lines rather than attempting to show the actual tidal curve. Lunar positions during the month are shown below the diagram and lunar phases above it. The tidal range is greater at the time of new and full moon than at first and last quarter. It is also greater when

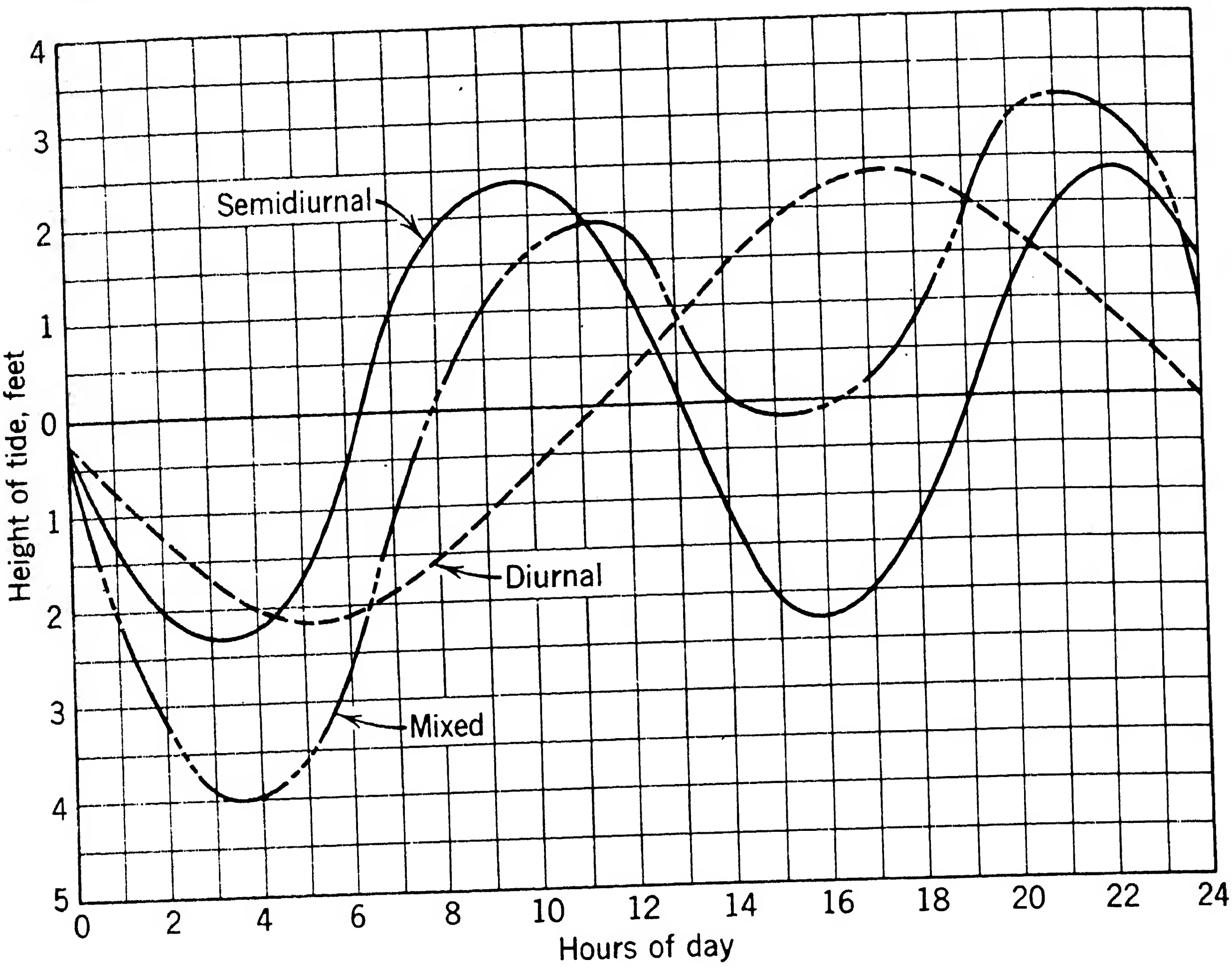


Figure 69. Types of tides.

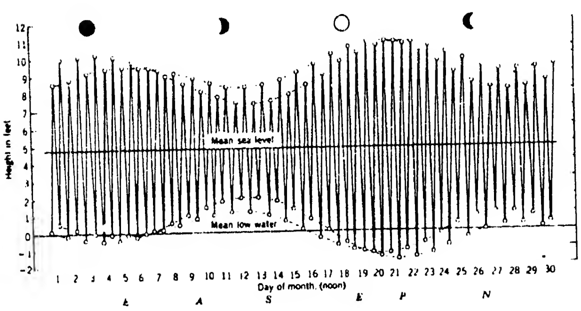


Figure 70. Tidal variations for a month at a port of embarkation.

the moon is at perigee, *P*, than when it is at apogee, *A*. At those times when the moon has its greatest north or south declination, shown by *N* and *S*, the diurnal inequality is quite marked, amounting to about 1.6 feet, whereas at points marked *E*, with the moon on the equator, this inequality is absent.

193. Tide Tables

From tidal observations covering extended periods and from careful analyses of all tide-producing factors, it is possible to predict the heights and times of occurrence of tides in various parts of the world. Such predicted data are published annually, in advance, by the U.S. Coast and Geodetic Survey in their tide tables. These tide tables, listed below, are items of military issue.

Tide Tables, Central and Western Pacific Ocean and Indian Ocean.
Tide Tables, East Coast North and South America Including Greenland.
Tide Tables, Europe and West Coast of Africa Including Mediterranean Sea.
Tide Tables, West Coast North and South America Including Hawaiian Islands.

These volumes given daily tidal data at a number of reference ports and stations and tables

of tidal differences and constants so that the daily values for any one of a large number of subordinate ports listed can be quickly computed from the data for a reference station. When such data are not available for ports under military control, it may be necessary to establish gaging stations.

194. Tidal Currents

The vertical rise and fall of the sea is accompanied by periodic horizontal movements of the water known as tidal currents. Such currents may have a considerable effect upon the actual execution of hydrographic surveys in any area, and they are of major importance to the port construction engineer. As such, they are treated in considerable detail in TM 5–360, to which reference is made for information on tidal current measurements. The U.S. Coast and Geodetic Survey publishes annual current tables. These include daily predictions of the times of slack water and the times and velocities or strength of flood and ebb currents for a number of waterways, together with differences for obtaining predictions for many other places. This same agency also publishes tidal current charts for a number of the major ports of the United States. For each port, a series of 12 charts shows, by means of arrows and figures, the direction and velocity of the current for each hour of the tidal cycle.

195. Tide Gages

Tide gages are instruments for measuring the height of the tide. They may be classified in two general groups: nonrecording gages, which require the presence of an attendant to observe and record the height of the tide at periodic intervals; and automatic or self-registering gages, which provide a continuous record of the variation of tide level with the passage of time and which will operate unattended for a number of days. The first group includes the staff gage and various types of float and pressure gages; the second consists of gages with a height-recording element which is usually actuated by a float and time-recording element motivated by clockwork. The record of the rise and fall of the tide may be shown on a graph, by printed figures, or by photographic means.

196. Engineer Applications of Tide Gaging

The chief applications of tide gaging in engineer operations are in connection with hydrographic surveys, dredging operations, or military subaqueous construction in an area. Short-term gaging records are required during the period of operations to permit reduction of measurements of water depths to a common datum, to control dredging, and to facilitate the scheduling of construction work. More rarely, observations will be extended over a longer period for the determination of tidal datums and for other purposes.

197. Tide Gages for Engineer Use

This manual describes only those gages which can be improvised or readily constructed for engineer use, or those recording types which are items of military issue or which may be made available for engineer use through government agencies. The recording gage ES 49–5 is the military-issue gage.

198. Tide-Gage Sites

Special care should be taken in the selection of a tide station. It should be located in a protected area. There should be a depth of approximately 5 feet (1.5 m) below the predicted lowest tide. The gage should be located where it will not be damaged by land or sea traffic.

199. The Staff Gage

a. Types. The staff gage is the simplest form of tide gage to construct, install and use. It consists of a vertical graduated staff securely fastened to a pole or other suitable support. The length of the staff and the location of the gage must be such that a positive reading of the water level may be obtained at any stage of the tide. The staff usually consists of a board 1 to 2 inches (25 to 50 mm) thick and from 4 to 6 inches (100 to 150 mm) in width. For painted gages, paint the board white and paint black graduations, in feet and tenths, on the white background. Salvaged level or stadia rods can be used for improvised gages where the tidal range is less than the length of the rod. Do not use a serviceable rod for this purpose. Painted graduations soon become illegible in harbor waters where they are defaced or stained by floating refuse and oil. The U.S.

Coast and Geodetic Survey uses vitrified enamel graduations baked on 3-foot sections of wrought-iron strips. The latter are secured to the staff with brass screws. Such gages are readily kept clean and legible. Staff gages should be screwed rather than nailed to the support so that they may be easily removed for cleaning or repainting or for storage when not in actual use. Figure 71 shows a typical tide-gage installation at high and low water at a point where the tidal range is great. The low water reading is 7.5 feet, the high water reading 41.2 feet, a difference of 33.7 feet. The three-pile cluster supports a pipe well for a recording float gage (para. 200) which is housed in the shelter atop the cluster. The illustration shows the method of support of the outside staff gage, the type of graduations used, and the access ladder from the boat to the float-gage house.

b. Stilling Devices. Stilling devices to permit accurate reading of the water level are desirable in locations exposed to wave action. The use of a stilling box is feasible where the tidal range is small (3 ft. or 1 meter or less). A glass tube stilling device is used where the range is greater. A glass tube about one-half of an inch in diameter is either secured to the face of the staff by spring clips or is fastened between two wood strips which carry the graduations. Water enters the glass tube through a very small hole or past a notched cork at the bottom. It will rise or fall rapidly enough to show the gradual fluctuations in the water level but the effect of wave action will be damped out. For ease of reading it is convenient to have a vertical red stripe painted on the back of the tube or blown into the glass. Above the water surface this stripe will show its true width. Below the surface, because of the refraction of light by the water, it will appear several times wider, making it easy to distinguish the surface line. Such a gage may be read from a distance with the aid of field glasses or a transit telescope. For ease in handling and repair, a gage of this type is made in 3-foot (1-meter) sections. Each pair of large tubes is joined by a small connecting tube passing through stoppers (fig. 72). A gage having a cross section of the shape shown in figure 73 is suitable for use in rapidly flowing tidal currents where an ordinary staff would be difficult to read because of the ripple resulting from the current striking the gage. As indicated by arrows in the figure, the side strips deflect the current and minimize the ripple effect.

200. The Float Gage

Float gages permit more accurate reading

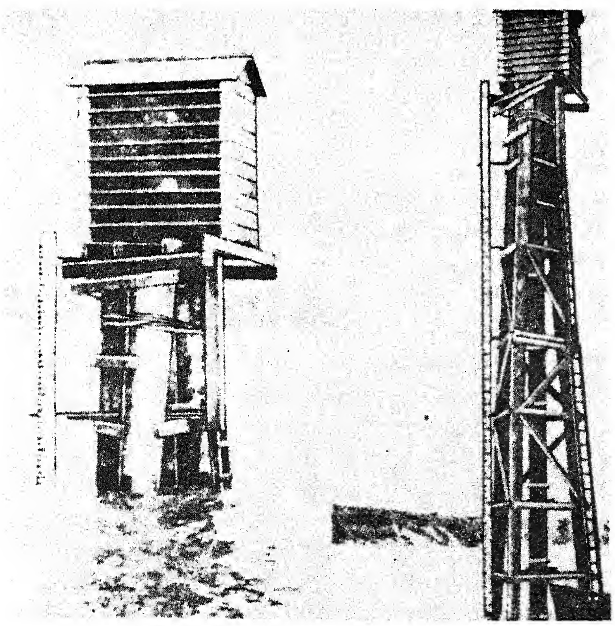


Figure 71. Tide-gage installation.

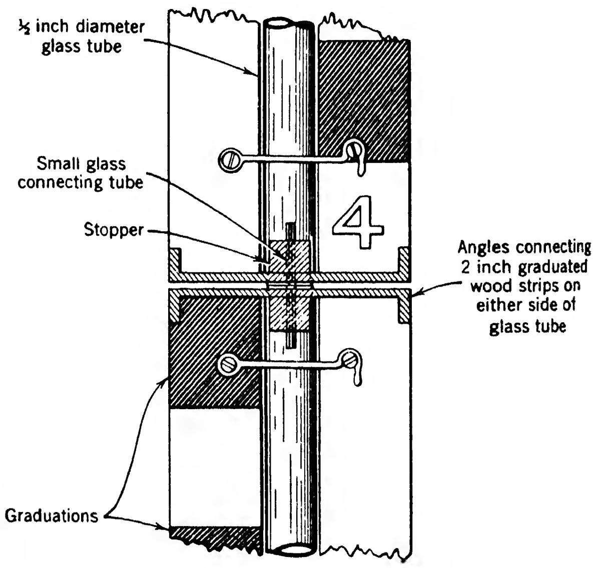


Figure 72. Glass-rod connection for staff gage.

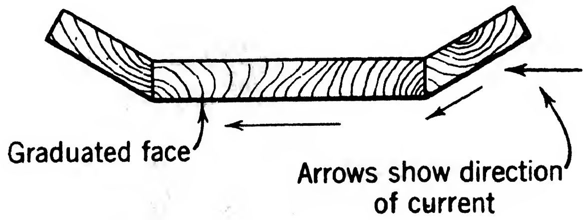


Figure 73. Staff-gage section for use in tidal currents.

of the height of the tide than do staff gages. They are used in exposed locations where the water is too rough for satisfactory staff-gage readings. Float gages of the recording type (para. 201) are often used in conjunction with staff gages. Thus, in figure 71 the gage shelter houses a recording float gage which gives a continuous record of the rise and fall of the tide. This gage can be read only by entering the gage house whereas the reading of the outside staff gage can be observed from a distance during the course of hydrographic operations in the area. The tape gage is the most commonly used form of nonrecording float gage. This is placed in a gage house located on a wharf, pier, or pile cluster and is situated over a vertical box or pipe float well which is connected to the sea by one or more openings located well below low water but above possible danger of clogging from shoaling. The main opening to the float chamber should be in the bottom rather than in the side for ease in cleaning. This opening should be small enough to dampen out the effect of the larger waves but large enough to permit the float to indicate rough water outside. Unless this is true, inflow or outflow may be at so slow a rate that there will be a time lag between water levels in the sea and in the well. A 1½-inch (38-mm) opening normally is satisfactory for a 12-inch (30-cm) pipe float well. Pipe wells are generally more durable than wooden box wells. In areas infested with teredos and other marine borers the wooden box wells should be sheathed with copper on the outside. Figure 74 shows a typical pipe float well (see also fig. 71). The gage is operated by a cylindrical float which rises and falls in the well with the rising and falling tide. The float is connected to a tape which passes over a pulley hung from the gage-house roof. When the tidal range is greater

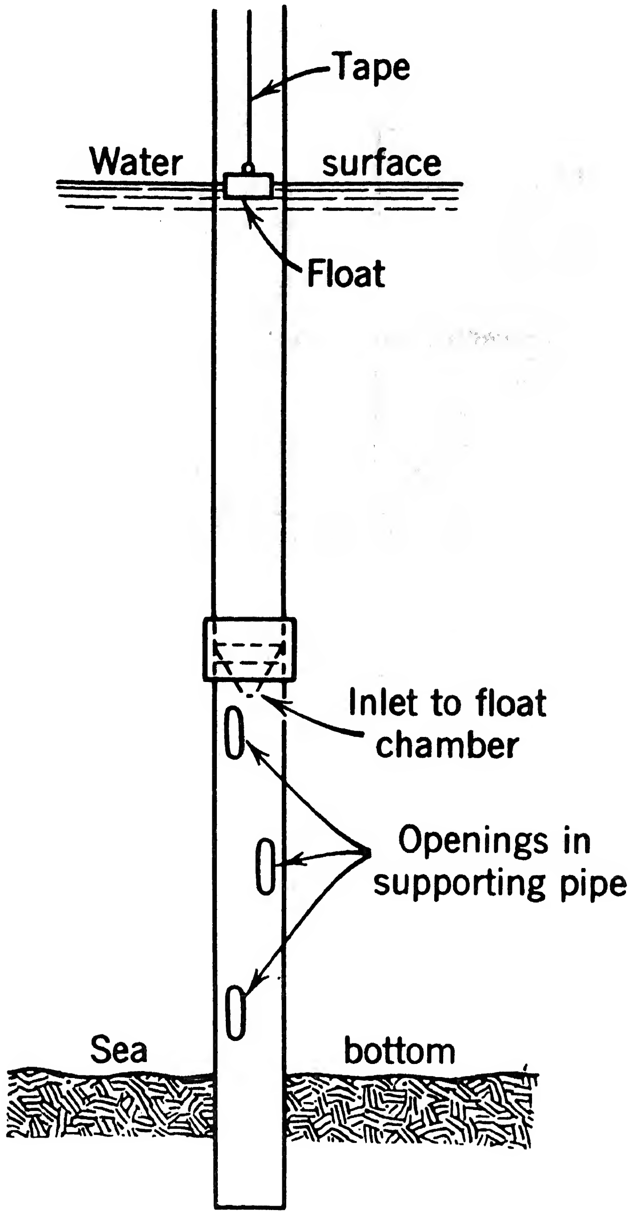


Figure 74. Pipe float well for float gage.

than the distance between the gage-house roof and its floor, the counterweight is hung from a movable pulley (fig. 75). An index pointer secured to the tape may move over a fixed scale, or, as in figure 75, the tidal height is read on the tape, graduated in this case, as the tape passes a fixed reading mark. The tape graduations should increase downward so that the readings will increase with a rising tide. The upper portion of the tape should preferably be

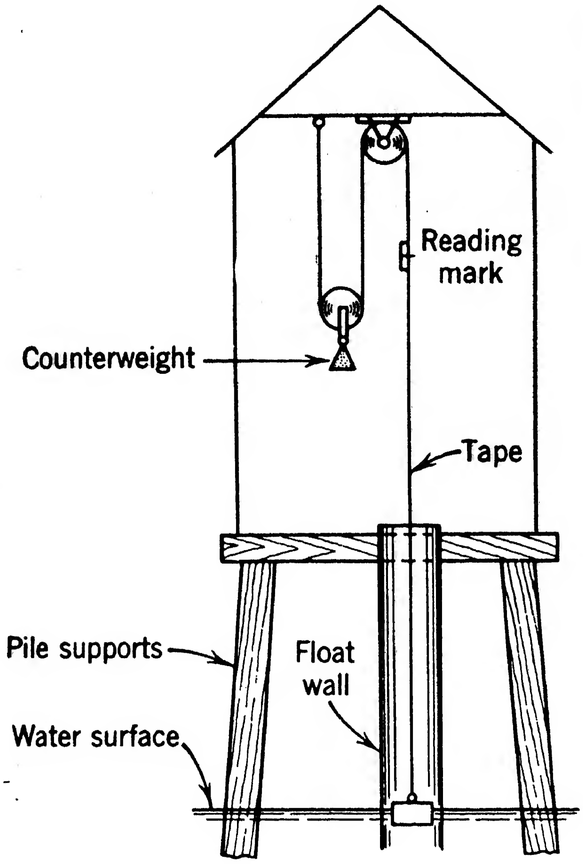


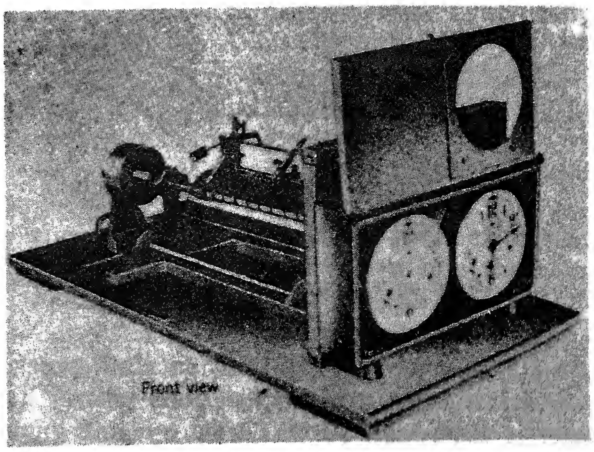
Figure 75. Gage house and tape gage.

of stainless steel for easy legibility, while a bronze alloy possessing greater durability should be used for the lower portion which is most exposed to salt water. For engineer operations covering a limited period a salvaged military-issued tape, kept well oiled, can be adapted to the purpose. A wire can be used in place of a tape when an index pointer and fixed scale are employed.

201. Automatic Tide Gages

Two principal types of the automatic tide gage are the standard automatic tide gage and the portable automatic tide gage.

a. Standard Automatic Tide Gage. This gage, used at tide stations where records are to be kept for a long period, is sheltered in a small gage house, about 6 feet (2m) square, located on a pier or other suitable support over a float well. The general arrangement is similar to the tape gage housing (fig. 75). The rise and fall



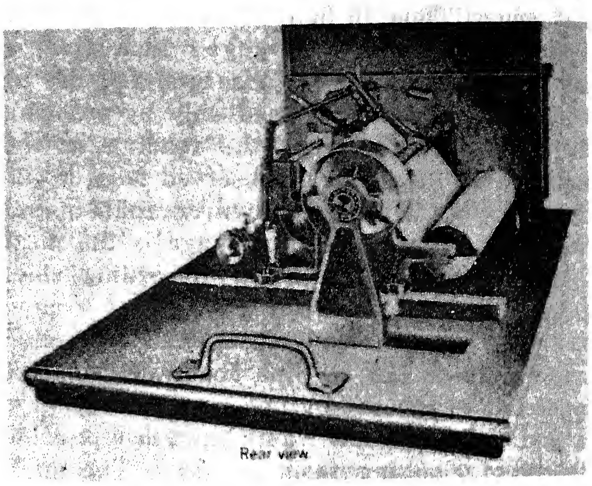


Figure 76. Standard automatic tide gage.

of the float is communicated by a wire to a worm screw on the gage which moves a pencil on a wide paper strip which is being moved forward between rollers by clockwork. The combined motions of pencil and paper produce a continuous graph showing the rise and fall of the tide. The time element of this gage consist of two 8-day clocks. One of these, the time clock, operates the device which shows the hour-marks on the record. The other, the motor clock, regulates the forward movement of the paper. Figure 76 shows front and rear views of the standard automatic tide gage. Any such instruments as may be requisitioned and issued for engineer use should be accompanied by complete instructions for the installation, operation, and maintenance of the gage. U.S. Coast and Geodetic Survey Special Publication

No. 196, Manual of Tide Observations, contains such information.

b. Portable Automatic Tide Gage. The portable gage was developed for use where a short series of observations is necessary for the reduction of soundings to a common datum. A hydrographic survey along an irregular stretch of coast, where both the tidal range and the times of high and low water vary rapidly, may require the installation of a considerable number of staff or automatic gages. The need for a gage which is readily portable and easily installed is evident. The portable gage (fig. 77) is much smaller than the standard gage, has a single clock movement, and differs from the standard gage in a number of other particulars. It is mounted directly on the top of the pipe which serves as the float well and is protected by an iron cover so that there is no need for a gage house. A typical installation of a portable automatic gage, with staff gage attached to the well pipe, is shown in figure 78. The gage cover, removed by the observer while taking the photograph, is shown on the approach platform which is held down by rocks since, at high tide at this station, the pproach platform is slightly below water level.

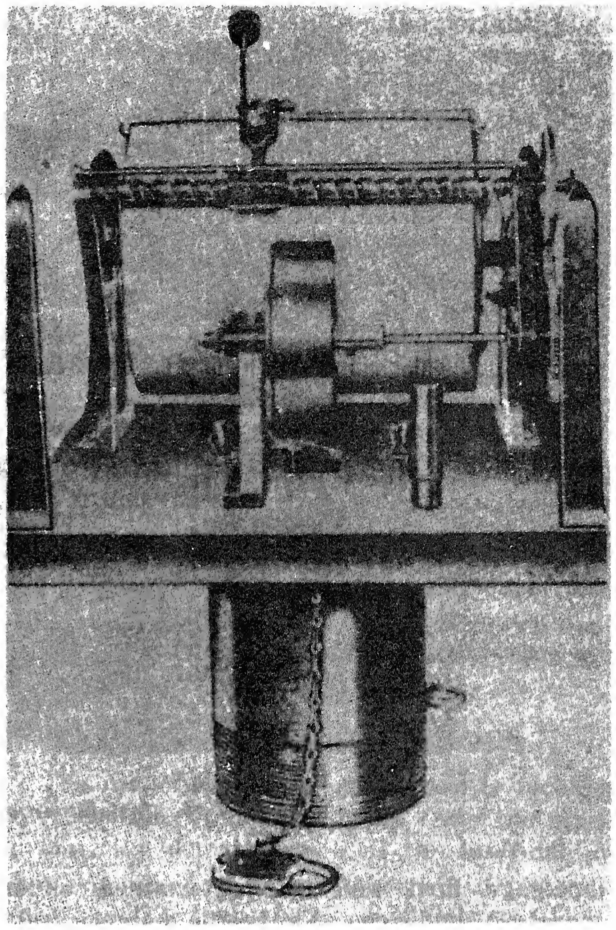


Figure 77. Portable automatic tide gage.

202. Location of Gages

For a hydrographic survey of an area, a primary tide station is centrally located in the area. This station should preferably be equipped with a standard automatic and a staff gage. In some localities, the tide occurs practically simultaneously and with the same range over a wide area. In such instances, the single station will suffice for survey purposes. In other localities, the tide changes rapidly in passing from point to point and secondary staff or portable automatic gages must be installed at frequent intervals to control the reduction of soundings in each locality. Each gage should be sheltered, if possible, from the action of heavy seas. The site selected must have sufficient water depth so that readings can be obtained at all stages of the tide. No general rule can be given for the frequency of installations of secondary gages. Temporary staff gages, connected by levels, should be installed at a number of points in the survey area during reconnaissance. Simultaneous readings at these gages will reveal whether or not there is a significant difference in stage from that recorded at the central station.

203. Installation of Gages

The staff gage presents little difficulty in installation. It may be fastened to a pile cluster (fig. 71) or braced in a vertical position in the water off ledges (fig. 78). It must be vertical and it must be so situated that a level rod, visible from an instrument position on shore, can be placed on a convenient reference mark on the gage. This is necessary so that differential levels can be run to permanent bench marks on shore to reference the zero of the gage and to permit periodic checking of the gage. In the installation of float gages, particular attention must be paid to plumbing the float-well pipe so that the float may move freely up or down. The plane of flotation of the

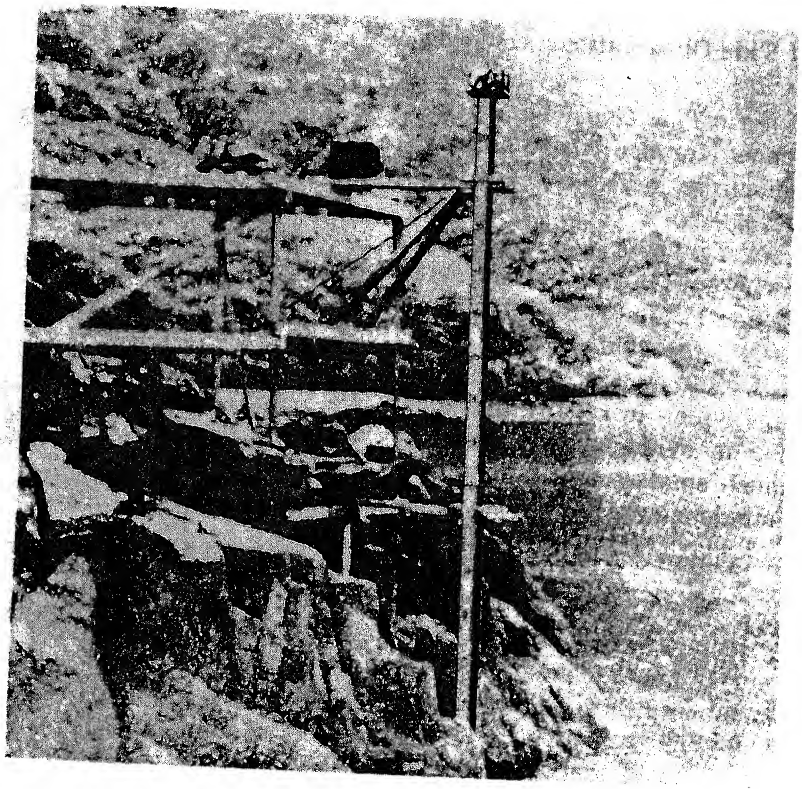


Figure 78. Installation of portable automatic tide gage.

tape-gage float must be determined under operating conditions, that is, the distance from the water surface to a graduation of the tape must be measured to calibrate the gage. It should be possible to hold a folding rule or short level rod on the reading mark and obtain a reading from a level on shore sighting through the gagehouse door so that differential levels can be run to a permanent bench mark. Artificial illumination and a target may be necessary for obtaining the rod reading in the gage house. The installation of automatic gages should follow instructions furnished with the gage.

204. Tide Records

a. For Nonregistering Gages. Records for nonregistering gages consist of sheets with parallel ruled columns in which are entered the times and the corresponding water levels.

b. For Standard Automatic Gage. The record paper for the standard automatic gage is supplied in rolls of plain paper some 13 inches (33 mm) wide and 66 feet (20 m) long, sufficient to record the tidal movement for a month. Once the tide curve has been marked upon the roll the record is known as a *marigram.* The height scale of the marigram is dependent upon the circumference of the float pulley and the pitch of the pencil screw. By varying these, a scale can be established which is appropriate to the range of tide to be recorded.

c. For Portable Automatic Gage. The record form for the portable automatic gage is furnished as a sheet with special rulings for the time and height scales. The sheet is wrapped around a drum driven by clockwork and provides a marigram covering a 48-hour period for each revolution of the drum. Different height scales may be arranged by using different gear ratios between the float-wire drum and the marking stylus.

205. Duties of Gage Attendants

a. For Nonregistering Gages. Read and record the height of the tide at prescribed intervals. When only the heights of high and low water are desired, take six readings at

10-minute intervals beginning about one-half hour before the predicted time of high or low water. When soundings are being taken in the area, periodic readings are to be taken throughout the day so that all soundings may be reduced to a common datum. Show the date, wind direction, and weather conditions on the daily record sheet.

b. For Self-Registering Gages. The attendant at a tide station equipped with an automatic gage should be thoroughly familiar with the operation of the gage and with its component parts. He should have some knowledge of mechanics and should have sufficient scientific training to appreciate the importance of careful observations. He should be able to make simple repairs and necessary adjustments, and be able to correct the majority of the operating difficulties which may arise. His duties may be classified by time periods.

(1) *Daily duties.* Inspect the tide station and make a note on the margin of the marigram, indicating the position of the recording pencil at the time by an arrow. The notation must include: the date and day of the week; the correct time as obtained from a reliable source; the corresponding time as indicated by the time clock of the gage; the tide height as taken from an outside staff gage or from a nonregistering float gage; data relative to the wind and weather; the observer's signature or initials. The information indicated, known as the *comparative note,* is not only shown on the marigram but is recorded on a daily form. When the standard automatic tide gage is used, these data are essential in adjusting gage readings to datum. Correct the gage clocks if necessary. Adjust the recording pencil for wear of the lead or adjust the stylus of the portable gage. Wind up the tension weight.

(2) *Semiweekly duties.* Wind the clocks. Although these have 8-day movements, semiweekly windings provide assurance against stopping which might result from an unavoidable interruption in the daily visits. Change the record sheet on the portable gage. Forty-eight hours of record are traced on the sheet for each revolution of the cylinder. Since the times of high and low water occur nearly an hour later each day, the tide curve traced on a sheet during several revolutions of the cylinder separated sufficiently for clarity unless the tidal range is no more than a few inches. Although a single sheet might serve for a week, the recommended practice is to change the sheet every 3 or 4 days.

(3) *Monthly duties.* Change the paper on the standard gage.

(4) *Occasional duties.* Clean the float well whenever the outside staff gage reading differs from the automatic gage reading. Make any repairs that are required.

206. Reduction of Tide-Gage Records

The reduction of tide-gage records includes the tabulation of values taken from the marigrams, the determination of the lunitidal intervals, and the computation of the various mean tidal levels, ranges, and datums. These reductions are covered in the following paragraphs.

207. Determination of Datum for Marigram Tide Curve

a. Standard Automatic Gage. The roll for the standard automatic gage is of plain paper. A short horizontal line intersecting the tide curve is produced each hour by the clock-driven mechanism of the gage, but there are no rulings to indicate the height of the tide at any instant above a given datum. It is necessary to draw an arbitrary datum line on the monthly marigram and obtain the relationship between this assumed datum and the datum of the staff gage at the tide station. This is done by scaling, to the nearest 0.05 foot (0.015 m), the distance from the assumed datum line to the tide curve at each arrow indicating a comparative note made during the daily inspection of the station. The entries are made on a form for comparative readings (fig. 79). The entries for the first three columns

COMPARATIVE READINGS BUCKSPORT TIDE STATION

(1) Date Year 19 day mo	(2) Time of Staff Reading h m	(3) Staff A feet	(4) Scale B feet	(5) Difference A-B feet	(6) Phase of Tide	(7) Remarks	
						Lat. 44° 46'N Long 67° 50'W	
1 Apr.	14 58	16.85	20.60	-3.75	R	Party Chief SFC D.M. Long	
2 "	16 00	16.20	19.90	-3.70	R	Tide Gage No 82 Scale 1:24	
3 "	11 00	20.55	24.40	-3.85	F	Tabulated by Pfc K.A.Smith 10 May 19	
4 "	9 56	22.10	25.90	-3.80	H	Preliminary scale setting of datum line=14.00ft.	
5 Apr	10 59	21.15	25.00	-3.85	F		
						Scale setting for 1 April to 30 April 1952	
						Sum of differences	-115.50
						Mea fferences	-3.85
27 Apr.	15 58	18.00	21.80	-3.80	R	Preliminary setting	14.00
28 "	10 56	18.05	21.85	-3.80	F	Setting for reduction to tide staff	10.15
29 "	10 00	20.50	24.35	-3.85	F	Constant for fixed datum	0.00
30 Apr.	11 00	20.30	24.20	-3.90	F	Setting for reduction to fixed datum	10.15

Figure 79. Sample notes for comparative tidal readings.

are taken directly from the observer's notes on the marigram.

(1) In column 1, enter the year, month, and day.

(2) In column 2, enter the time of the staff-gage reading.

(3) In column 3, enter the mean staff-gage reading at this time.

(4) Select, arbitrarily, a convenient scale-setting for the datum line drawn on the marigram and enter this value in the remarks column. This preliminary setting should preferably be of such a value that the scale readings from the tide curve will be somewhat less than the corresponding staff readings. This is not essential, however. In the example, the scale readings are nearly 4 feet greater than the staff readings.

(5) Place the scale on the marigram with the preliminary setting in coincidence with the datum line and read the height of the curve at the point to which the corresponding staff reading refers. Take the reading at each such point to the nearest 0.05 foot. Enter the values in column 4.

(6) Enter the phase of the tide as shown by the marigram in column 6. Here *H* denotes high water, *L* indicates low water; *R* and *F* represent a rising or falling tide respectively.

(7) Enter the difference between columns 3 and 4 in column 5. These differences

should be approximately equal unless the gage has been adjusted during the month. Any values which differ materially from the apparent mean must be circled as rejected and omitted from the computation of the mean.

(8) In the remarks column, show the period for which the computation is being made, enter the sum of the differences in column 5 and divide by the number of days in the period to obtain the mean difference. This value, added algebraically to the preliminary setting, will give the scale reading for the datum drawn on the marigram so that readings from the tide curve will be reduced to the zero of the tide staff. If the zero of the tide staff does not conform to the datum being used for hydrographic surveys in the area, it will be necessary to apply a further "constant for fixed datum."

(9) The setting for reduction to fixed datum thus obtained is to be applied to all readings of hourly heights taken from the marigram.

(10) If the gage has been adjusted during the month, for example, by the installation of a new float wire on 10 April, there may be an appreciable difference in the values in column 5 for the first 10 days and for the last 20 days. In this case, a setting for reduction to fixed datum would be computed for each of the two periods.

b. Portable Automatic Gage. In the portable-gage record, the heights are taken directly from the horizontal rulings of the sheet. The zero ruling is assumed to correspond to the zero of the staff gage unless otherwise noted. The heights as indicated by the horizontal rulings are compared with the staff-gage readings as recorded in the comparative notes and allowance is made for any difference when the record is tabulated. The portable gage is designed to be reset when necessary so that the scale and staff readings may be kept in agreement.

208. Hourly Heights

Tidal heights are taken for each hour from the marigram by setting the scale to read the reduction to fixed datum, as determined by the method described in preceding paragraph, and obtaining the scale reading for the hour on the tidal curve. These values are tabulated in a form for hourly heights. This form (fig. 80) can be executed on the standard field-note form by ruling extra lines at the top and bottom of the page to provide for appropriate headings and tabulations of the tidal heights for each hour of the day. Values for 7 days can be shown on each page and still provide space at the right for appropriate remarks. Sums of values are taken both horizontally and vertically to provide a check. Mean tidal height for a period of record is obtained by dividing the totals for the period by the number of hours in that period.

209. High and Low Water Tabulation

Tabulations of the daily times and heights of high and low waters is made for each calendar month using columns 1, 3, 4, 7, 8, and 9 of the form shown in figure 81. This figure shows values for the last 6 days of a month. Several such sheets will be required for a month's record, the sums of the values entered in columns 7 and 8 being carried forward from previous sheets so that values of the mean range and mean tide level (para. 211) may be computed. The times of high and low waters are read from the marigram to the nearest tenth-hour. The heights are scaled from the marigram in the same manner as are the values of hourly heights (para. 208). This same form is used for the determination of lunitidal intervals (para. 210), columns 2, 5, and 6 being used for this purpose.

210. Determination of Lunitidal Intervals

a. Greenwich and Local Lunitidal Intervals. The Greenwich lunitidal interval for a place is the actual difference in time between the transit of the moon over the meridian of Greenwich and the occurrence of the following high or low water at the place. Separate intervals are obtained for high and low waters. The local lunitidal interval for a place is the difference in time between the transit of the

Day & Mo. Hour 0	1 Mar. 15.1 Ft.	2 15.5 Ft.	3 15.4 Ft.	4 13.9 Ft.	5 12.0 Ft.	6 9.0 Ft.	7 6.6 Ft.	Sum 87.5 Ft.	Tides: Remarks: Hourly Heights		
1	14.4	15.7	16.6	15.9	14.8	12.1	9.5	99.0	Station: Portsmouth		
2	13.5	15.4	17.0	17.3	17.1	15.1	12.8	108.2	Lat. 44° 50′ N		
3	12.5	14.8	16.9	17.9	18.6	17.5	15.8	114.0	Long. 68° 10′ W		
4	11.7	14.0	16.5	17.8	19.2	19.0	18.0	116.2	Party Chief SFC Long		
5	11.6	13.3	15.7	17.3	19.1	19.6	19.4	116.0	Tide Gage No. 85		
6	12.3	13.2	14.9	16.4	18.5	19.5	19.8	114.6	Scale 1:24		
7	13.7	13.7	14.6	15.5	17.4	18.7	19.5	113.1	Tabulated by Pfc. Smith		
8	15.4	15.0	15.0	15.0	16.3	17.6	18.6	112.9			
9	17.6	16.5	15.9	15.2	15.6	16.3	17.1	114.2			
10	19.2	18.2	17.2	16.0	15.8	15.6	15.9	117.9			
11	20.1	19.4	18.5	17.2	16.6	15.6	15.1	122.5			
12	19.9	19.8	19.4	18.4	17.7	16.3	15.4	126.9			
13	19.0	19.3	19.7	19.2	18.7	17.5	16.2	129.6			
14	17.3	18.0	18.9	19.2	19.5	18.4	17.3	128.6			
15	15.0	15.9	17.3	18.2	19.4	19.0	18.3	123.3			
16	12.2	13.1	14.8	16.3	18.1	18.6	18.9	112.0			
17	10.3	10.5	11.8	13.6	15.9	17.1	18.4	97.7			
18	9.5	8.5	9.2	10.5	13.0	14.7	16.8	82.2			
19	9.7	7.8	7.4	7.8	9.8	11.5	14.1	68.1			
20	10.5	8.3	6.7	6.1	7.0	8.1	10.9	57.6			
21	11.8	9.5	7.5	5.8	5.3	5.6	7.8	53.3			
22	13.4	11.4	9.1	7.0	5.1	4.2	5.4	55.6			
23	14.8	13.6	11.4	9.1	6.6	4.6	4.2	64.3			
Sum	340.5	340.4	347.5	346.6	357.1	351.2	352.0	2435.3			

Figure 80. Sample notes for hourly heights of tide.

moon over the local meridian and the time of the following high or low water at the place. The advantage of the Greenwich lunitidal intervals lies in the fact that the difference in the times of high or low water at two places is determined directly from the difference in their Greenwich lunitidal intervals. Mean local high water intervals for many places are published in the annual tide tables.

b. Determination of the Greenwich Lunitidal Intervals. The form used for recording high and low waters (fig. 81) provides for the computation of high and low water lunitidal intervals.

(1) Take the times of the moon's upper and lower transits at Greenwich for each day from the American Ephemeris and Nautical Almanac. In this volume the times are given in hours and minutes of Greenwich civil time. Convert the values to hours and tenths and enter them in column 2 (fig. 81). In general, there will be both an upper and a lower transit each day. Enclose the times of lower transit in parentheses to distinguish them from the times of upper transit.

(2) From the time of each high and low water given in columns 3 and 4, subtract the time of the first preceding transit of the moon and enter the difference in the appropriate column

HIGH AND LOW WATERS AND LUNITIDAL INTERVALS – Seattle Tide Station											
1	2	3	4	5	6	7	8		9		
Date Year 1928	Moon's Transits	Time High	of Low	Lunitidal High	Interval Low	Height High	of Low	Remarks:			
Day Mo.	G.C.T. hr. dec.	Water hr. dec.	Water hr. dec.	Water hr. dec.	Water hr. dec.	Water feet	Water feet	Lat. 47° 37' N			
Brought forward				243.7	537.7	879.5	513.3	Long. 122° 20' W			
26 Jan.	(3.0)	8.0	0.9	(5.0)	10.3	19.4 ✓	6.3 ✓	Time meridian 120° (8ʰ) W			
	15.5	19.0	14.0	3.5	(11.0)	16.8	12.4	Tabulated by: R.D.M.			
27	(3.9)	8.5	1.6	(4.6)	10.1	19.5 ✓	7.1 ✓	Reduced by: F.G.L.			
	16.3	20.2	15.0	3.9	(11.1)	16.4	11.6	Checked by: W.S.L.			
28	(4.7)	9.0	2.3	(4.3)	10.0	20.0 ✓	9.0 ✓				
	17.1	21.5	16.0	4.4	(11.3)	16.1	10.7				
29	(5.5)	9.9	3.9	(4.4)	10.2	20.1 ✓	11.2				
	17.9	23.2	16.9	5.3	(11.4)	16.0	9.5 ✓				
30	(6.3)	10.7	4.3	(4.4)	10.4	19.5 ✓	13.2				
	18.8	—	17.8	—	(11.5)	—	8.1 ✓				
31	(7.2)	0.9	6.0	6.1	11.2	16.6	14.7				
	19.7	11.4	18.4	(4.2)	(11.2)	19.9 ✓	7.3 ✓				
								HHW	LLW		
Sums				293.8^{59}	667.4^{60}	1079.8^{59}	634.4^{60}	599.7^{31}	204.7^{29}	Sums	
Unreduced intervals				4.98	11.12	18.30	10.57	19.35	7.06	Means	
Greenwich intervals				0.56	6.70	10.57			Mn	DHQ	DLQ
Local intervals				4.54	10.68	7.73	Mn	Observed	7.73	1.05	3.51
						14.44	MTL	Factor	1.004	0.85	0.85
								Corrected	7.76	0.89	2.98

Figure 81. Sample notes for tabulation of high and low waters and lunitidal intervals.

(5 or 6), and on the same line as the tide from which it was obtained. In case the time of high or low water is nearly the same as that of the moon's transit, take the transit which precedes the tide by about 12 hours. In no case, however, is the same transit of the moon to be used for two consecutive high waters or for two consecutive low waters. The lower transit of the moon applies to both high and low water, just as the upper transit does. When the time of the moon's transit is on one day and the following high or low water is on the next day, increase the time of this tide by adding 24 hours to it before subtracting the time of transit. Enclose intervals obtained from the moon's lower transits in parentheses. For illustration, take the data for 27 January (fig. 81). From column 3 we have 8.5 hours as the time of the first high water on this date. Subtract the time of the preceding transit (3.9, col. 2) to obtain the value of 4.6 hours which is entered in column 5. Since this value is obtained from a lower transit of the moon it is enclosed in parentheses. The first low water on 27 January occurred at 1.6 hours (col. 4). The preceding transit of the moon occurred at 15.5 hours on 26 January (col. 2). Therefore add 24

hours to the time of low water to obtain 25.6 hours before subtracting the 15.5 hours. This subtraction gives 10.1 hours as the low water interval which is entered in column 6. Similarly, for the second high water on this day we have 20.2 (col. 3) — 16.3 (col. 2) = 3.9 (col. 5). For the second low water, we have 15.0 (col. 4) — 3.9 (col. 2) = 11.1 (col. 6). This latter value is enclosed in parentheses since it was determined from a lower transit.

(3) Add the high and low water intervals separately for the calendar month, carrying the totals forward from page to page of the notes. Compute the mean values of the intervals to two decimal places.

(4) In the computations of (3) above, the moon's transits are given in Greenwich civil time while the times of high and low waters are entered in the standard time of the place of observation. To obtain the Greenwich lunitidal intervals it is necessary to correct the intervals obtained above by applying the difference in time between Greenwich and the time meridian of the place, this correction being added for west longitude and subtracted for east longitude. If this corrected value for either interval exceeds 12.42 hours (the semidiurnal lunar period), it is to be reduced by this value. For the example of figure 81, standard time in Seattle is 8 hours earlier than (west of) Greenwich time. Adding 8 hours to the unreduced high-water interval of 4.98 hours, we obtain 12.98 hours. Subtracting 12.42 hours from this, the Greenwich lunitidal interval of 0.56 hours is obtained. The low water interval of 6.70 is computed in a similar manner.

c. Determination of Local Lunitidal Intervals. To change from Greenwich to local intervals, it is necessary to apply a correction equal to the time required for the moon to pass from the meridian of Greenwich to the local meridian of the place of observation. Table III gives this correction for every degree of longitude from 1° to 180°, and the value for each minute from 1′ to 60′. In converting from the Greenwich interval to the local interval, this correction is to be subtracted for a place of observation in west longitude and added if the place is in east longitude. In order that the local intervals may be positive and less than half the lunar day, the semidiurnal lunar period of 12.42 hours is to be added or subtracted as may be necessary. Referring to the tide station of figure 81, in longitude 122° 20′ West, the computation of the local lunitidal intervals is made as follows:

(1) From table III, opposite longitude 122°, take a correction of 8.418 hours, and opposite longitude 20′, take a correction of 0.023 hours. The total correction is 8.441 hours.

(2) Taking the Greenwich high water lunitidal interval of 0.56 hours, add the semidiurnal lunar period of 12.42 hours to obtain a value of 12.98 hours. Since the tide station is in west longitude, subtract the table III correction from this value. The difference, 4.54 hours (fig. 81), is the local high water lunitidal interval.

(3) Compute the local low water lunitidal interval, 10.68 hours as shown in figure 81, in a similar manner.

211. Height Reductions

The tabulation of high and low water data (fig. 81) furnishes information for monthly computations of tidal planes and ranges. *Mean high water* (*MHW*) and *mean low water* (*MLW*) for the month are obtained from the summation of all the high waters and the summation of all the low waters. These totals are divided by the total number of observations in each case to obtain the mean values. In figure 81, mean high water is 18.30 feet (col 7); mean low water is 10.57 feet (col 8). The *mean range* (*Mn*) is obtained by subtracting the mean low water value from the value for mean high water (7.73 feet, fig. 81). The *mean tide level* (*MTL*), also known as *half-tide* level, is obtained by taking half the sum of the values for mean high and mean low water (14.44

Table III. Table for Reducing Greenwich Intervals to Local Intervals

Longitude	Correction	Longitude	Correction	Longitude	Correction	Longitude	Correction	Longitude	Correction	Longitude	Correction	Longitude	Correction
′	*Hour*	′	*Hour*	°	*Hour*	°	*Hour*	°	*Hour*	°	*Hour*	°	*Hour*
1	0.001	36	.041	6	.414	41	2.829	76	5.244	111	7.659	146	10.074
2	.002	37	.043	7	.483	42	2.898	77	5.313	112	7.728	147	10.143
3	.003	38	.044	8	.552	43	2.967	78	5.382	113	7.797	148	10.212
4	.005	39	.045	9	.621	44	3.036	79	5.451	114	7.866	149	10.281
5	.006	40	.046	10	.690	45	3.105	80	5.520	115	7.935	150	10.351
6	.007	41	.047	11	.759	46	3.174	81	5.589	116	8.004	151	10.420
7	.008	42	.048	12	.828	47	3.243	82	5.658	117	8.073	152	10.489
8	.009	43	.049	13	.897	48	3.312	83	5.727	118	8.142	153	10.558
9	.010	44	.051	14	.966	49	3.381	84	5.796	119	8.211	154	10.627
10	.012	45	.052	15	1.035	50	3.450	85	5.865	120	8.280	155	10.696
11	.013	46	.053	16	1.104	51	3.519	86	5.934	121	8.349	156	10.765
12	.014	47	.054	17	1.173	52	3.588	87	6.003	122	8.418	157	10.834
13	.015	48	.055	18	1.242	53	3.657	88	6.072	123	8.487	158	10.903
14	.016	49	.056	19	1.311	54	3.726	89	6.141	124	8.556	159	10.972
15	.017	50	.058	20	1.380	55	3.795	90	6.210	125	8.625	160	11.041
16	.018	51	0.059	21	1.449	56	3.864	91	6.279	126	8.694	161	11.110
17	.020	52	.060	22	1.518	57	3.933	92	6.348	127	8.763	162	11.179
18	.021	53	.061	23	1.587	58	4.002	93	6.417	128	8.832	163	11.248
19	.022	54	.062	24	1.656	59	4.071	94	6.486	129	8.901	164	11.317
20	.023	55	.063	25	1.725	60	4.140	95	6.555	130	8.970	165	11.386
21	.024	56	.064	26	1.794	61	4.209	96	6.624	131	9.039	166	11.455
22	.025	57	.066	27	1.863	62	4.278	97	6.693	132	9.108	167	11.524
23	.026	58	.067	28	1.932	63	4.347	98	6.762	133	9.177	168	11.593
24	.028	59	.068	29	2.001	64	4.416	99	6.831	134	9.246	169	11.662
25	.029	60	.069	30	2.070	65	4.485	100	6.900	135	9.315	170	11.731
26	.030			31	2.139	66	4.554	101	6.969	136	9.384	171	11.800
27	.031			32	2.208	67	4.623	102	7.038	137	9.453	172	11.869
28	.032			33	2.277	68	4.692	103	7.107	138	9.522	173	11.938
29	.033			34	2.346	69	4.761	104	7.176	139	9.591	174	12.007
30	.035	°	*Hour*	35	2.415	70	4.830	105	7.245	140	9.660	175	12.076
31	.036	1	0.069	36	2.484	71	4.899	106	7.314	141	9.729	176	12.145
32	.037	2	.138	37	2.553	72	4.968	107	7.383	142	9.798	177	12.214
33	.038	3	.207	38	2.622	73	5.037	108	7.452	143	9.867	178	12.283
34	.039	4	.276	39	2.691	74	5.106	109	7.521	144	9.936	179	12.352
35	.040	5	.345	40	2.760	75	5.175	110	7.590	145	10.005	180	12.421

feet, fig. 81). The levels of *mean higher high water* (*MHHW*) and *mean lower low water* (*MLLW*) are used in hydrographic surveys on the Pacific Coast. Using the form of figure 81, the higher of the two high waters and the lower of the two low water of each day of the month are indicated by a checkmark (fig. 81). If the two high or two low waters on the same day are equal, either may be selected as the higher high or lower low water. When only one high or one low water occurs on a calendar day, because one of the tides occurs after midnight and therefore on the next calendar day, the single tide is to be checked if the tide just above it is unchecked (30 January, fig. 81); otherwise, it should not be checked. If, however, the tide has become diurnal and only one high and one low water occur during the tidal day, these should both be checked. Summations of the checked heights are made separately for the high and low waters and the sums are entered to the right of the bottoms of the columns (599.7 and 204.7, fig. 81.) The number of observations is then entered in small figures just above each sum and divided into the corresponding sum to

obtain the mean higher high water and the mean lower low water, results being carried to two decimal places (19.35 feet, and 7.06 feet, fig. 81). The *diurnal high water inequality* (*DHQ*) is obtained by subtracting the value for mean high water from the value for mean higher high water (19.35 — 18.30 = 1.05 feet, fig. 81). Similarly, the *diurnal low water inequality* (*DLQ*), is obtained by subtracting the value for mean lower low water from the value for mean low water (10.57 — 7.06 = 3.51 feet) (fig. 81).

212. Correction for Longitude of Moon's Node

As indicated in paragraph 186, when tidal observations continue over less than a 19-year period, values of mean levels and ranges must be corrected for the longitude of the moon's node. When the observations extend over less than a year, the diurnal inequalities must be corrected for the effect of variations in the declination of the sun.

a. Table IV contains values of the *factor for mean range*, *F* (*Mn*), by which the observed tidal range must be multiplied to obtain its mean value. The values given have been computed for the middle of the year indicated but change so slowly that the tabular value for the year may be used for any month in the year without appreciable error. This factor *F*(*Mn*) varies not only with the year of observation but also upon the relation of the diurnal to the semidiurnal wave in the locality. This relationship is expressed approximately by the formula—

$$\frac{2(DHQ+DLQ)}{Mn}$$

For Atlantic Coast stations from Maine to Florida, the ratio is generally small and may be assumed to be less than 0.2 if values for *DHQ* and *DLQ* have not been computed. For Gulf Coast stations between Key West and the Rio Grande, the mean tide range is small and the correction factor may be omitted. The ratio should be computed for Pacific Coast stations and for stations in foreign waters. If the ratio is greater than 2.0, no correction need be applied to the mean range. Uncorrected values for *DHQ*, *DLQ*, and *Mn* may be used in computing the ratio. In figure 81, the correction factor for the year of observation was 1.004. Multiplying this by the observed value of *Mn*, the corrected value of 7.76 feet is obtained.

Table IV. Factor F (Mn) For Reducing the Observed Range of Tide to Its Mean Value

Mn / Year \ $z(DHQ + DLQ)$	0.0 to 0.2	0.3 to 0.4	0.5 to 0.6	0.7 to 0.8	0.9 to 1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
1951	1.028	1.027	1.025	1.023	1.020	1.017	1.013	1.008	1.001	0.995
1952	1.025	1.024	1.022	1.020	1.017	1.014	1.010	1.006	1.001	.996
1953	1.017	1.016	1.015	1.014	1.012	1.010	1.007	1.004	1.001	.997
1954	1.008	1.008	1.008	1.007	1.005	1.004	1.003	1.002	1.000	.998
1955	.998	.998	.998	.998	.998	.999	.999	1.000	1.000	1.000
1956	.987	.987	.988	.990	.992	.994	.996	.998	1.000	1.002
1957	.979	.980	.981	.983	.985	.988	.991	995	.999	1.004
1958	.972	.974	.976	.978	.981	.984	.988	.993	.998	1.005
1959	.971	.972	.974	.976	.979	.983	.987	.992	.999	1.006
1960	.971	.972	.974	.976	.979	.983	.987	.992	.999	1.006
1961	.974	.975	.977	.979	.982	.985	.988	.993	.998	1.005
1962	.979	.981	.982	.984	.986	.989	.992	.995	.999	1.004
1963	.989	.989	.989	.990	.992	.994	.996	.998	1.000	1.002
1964	.999	.999	.999	.999	.999	.999	1.000	1.000	1.000	1.000
1965	1.009	1.009	1.008	1.007	1.006	1.005	1.004	1.002	1.000	.998
1966	1.018	1.017	1.016	1.014	1.012	1.010	1.007	1.005	1.001	.996
1967	1.025	1.024	1.022	1.020	1.017	1.015	1.011	1.006	1.001	.996
1968	1.029	1.028	1.026	1.023	1.020	1.017	1.013	1.008	1.001	.995
1969	1.030	1.029	1.027	1.023	1.020	1.017	1.013	1.008	1.001	.995
1970	1.027	1.026	1.024	1.022	1.019	1.016	1.012	1.007	1.001	.996
1971	1.022	1.021	1.019	1.017	1.015	1.013	1.009	1.005	1.001	.996
1972	1.013	1.013	1.012	1.011	1.009	1.008	1.006	1.003	1.001	.998
1973	1.004	1.004	1.004	1.003	1.002	1.002	1.001	1.001	1.000	.999
1974	.994	.993	.993	.994	.996	.997	.998	.999	1.000	1.001
1975	.983	.984	.985	.987	.989	.992	.994	.996	.999	1.003
1976	.976	.978	.979	.981	.983	.986	.989	.994	.999	1.004
1977	.972	.973	.975	.977	.980	.984	.988	.993	.998	1.005
1978	.970	.971	.973	.975	.978	.983	.987	.992	.999	1.006
1979	.970	.972	.974	.976	.979	.983	.987	.992	.999	1.006
1980	.976	.977	.978	.980	.983	.986	.989	.994	.999	1.004

b. Table V contains values of the factor F_1, computed for the middle of each calendar month. The diurnal inequalities *DHQ* and *DLQ* are multiplied by this factor to obtain their mean values. The table also contains the mean of all monthly factors for the year. These values are used to correct yearly averages of *DHQ* and *DLQ* rather than the monthly values. For the observations shown in figure 81, the F_1 factor for January of that year was 0.85. The corrected values of *DHQ* and *DLQ* are, accordingly, 0.89 feet and 2.98 feet. Values of the two factors are shown in tables IV and V through the year 1980. Values for years subsequent thereto may be obtained from the Director, U.S. Coast and Geodetic Survey, Washington, D. C.

Table V. Factor F_1 for Correcting DHQ and DLQ

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
1951	0.78	0.91	1.04	0.97	0.82	0.76	0.78	0.91	1.06	0.97	0.82	0.76	0.882
1952	.79	.92	1.06	.98	.83	.76	.79	.92	1.07	.99	.83	.77	.892
1953	.80	.94	1.08	1.01	.85	.78	.81	.95	1.11	1.02	.86	.79	.917
1954	.83	.97	1.13	1.04	.88	.80	.84	.98	1.16	1.07	.89	.82	.951
1955	.86	1.01	1.19	1.09	.91	.84	.87	1.03	1.23	1.12	.93	.85	.994
1956	.90	1.06	1.26	1.16	.96	.87	.91	1.09	1.31	1.19	.97	.89	1.048
1957	.94	1.12	1.34	1.23	1.00	.91	.95	1.15	1.39	1.26	1.02	.92	1.102
1958	.98	1.18	1.42	1.29	1.04	.94	.99	1.20	1.46	1.32	1.05	.95	1.152
1959	1.01	1.22	1.48	1.33	1.07	.96	1.01	1.22	1.50	1.34	1.06	.96	1.180
1960	1.01	1.23	1.49	1.34	1.07	.96	1.00	1.22	1.49	1.33	1.05	.95	1.178
1961	1.00	1.21	1.45	1.31	1.05	.94	.98	1.18	1.43	1.28	1.02	.92	1.148
1962	.97	1.16	1.38	1.25	1.01	.91	.94	1.12	1.35	1.21	.98	.89	1.098
1963	.93	1.10	1.30	1.18	.96	.87	.90	1.07	1.27	1.14	.93	.85	1.042
1964	.89	1.04	1.22	1.11	.92	.83	.86	1.01	1.19	1.08	.89	.82	.988
1965	.85	.99	1.15	1.06	.88	.80	.83	.97	1.13	1.03	.86	.79	.945
1966	.82	.95	1.10	1.02	.85	.78	.81	.94	1.09	1.00	.84	.77	.914
1967	.80	.93	1.07	.99	.83	.76	.79	.91	1.06	.98	.82	.76	.892
1968	.79	.91	1.05	.97	.82	.76	.78	.90	1.05	.96	.81	.75	.879
1969	.78	.90	1.04	.96	.82	.75	.78	.90	1.04	.97	.81	.75	.875
1970	.78	.91	1.05	.97	.82	.76	.79	.91	1.06	.98	.82	.76	.884
1971	.79	.92	1.06	.99	.84	.77	.80	.93	1.09	1.00	.84	.78	.901
1972	.81	.95	1.10	1.02	.86	.79	.82	.96	1.13	1.04	.87	.80	.929
1973	.84	.98	1.15	1.06	.89	.82	.85	1.00	1.19	1.09	.90	.83	.967
1974	.87	1.03	1.21	1.12	.93	.85	.89	1.05	1.26	1.15	.94	.87	1.014
1975	.91	1.09	1.29	1.19	.98	.89	.93	1.11	1.34	1.22	.99	.90	1.070
1976	.95	1.14	1.38	1.26	1.02	.92	.97	1.17	1.42	1.28	1.03	.94	1.123
1977	.99	1.20	1.45	1.31	1.06	.95	1.00	1.21	1.48	1.33	1.06	.96	1.167
1978	1.01	1.23	1.49	1.34	1.08	.96	1.01	1.22	1.50	1.34	1.06	.96	1.183
1979	1.01	1.22	1.48	1.33	1.07	.96	1.00	1.20	1.47	1.31	1.04	.94	1.169
1980	.99	1.19	1.43	1.29	1.03	.93	.97	1.16	1.40	1.25	1.00	.91	1.129

213. Tidal Datums

Tidal datums are specific tide levels which are used as surfaces of reference for depth measurements in the sea and as a base for the determination of elevation on land. Many different datums have been used, particularly for leveling operations, and the chief of a survey party should be careful to ascertain the datum used in the establishment of bench marks from which he may have occasion to work. The principal tidal datums are—*Mean sea level; mean low water; mean lower low water;* and *mean low water springs.*

214. Mean Sea Level

Mean sea level (*MSL*) is the datum for the first-order level net of the United States and is increasingly used as the base for general leveling operations. It may be defined as the average height of the sea for all stages of the tide as determined from long period observations. It is obtained by averaging the hourly heights as tabulated on the form shown in figure 80. The heights on this form are added both horizontally and vertically. The total sum covering 7 days of record is entered in the lower-right corner of the page. The mean for each calendar month is found by combining all daily sums for the month and dividing by the total number of hours in the month. The monthly mean, to two decimal places, is entered on that sheet which includes the record for the last day of the month. Yearly means are determined from the monthly means and a mean is taken of all yearly means for the period of record. Three or more years of record should be used for a good determination of sea level. The actual value varies somewhat from place to place but

this variation is small. The station used for *MSL* determinations should be on the open coast or on the shore of bays or harbors having free access to the sea. Stations on tidal rivers at some distance from the open sea will have a *mean river level* which is higher than mean sea level because of the river slope. It should be noted that mean sea level is not identical with mean tide level (*MTL*). The latter is derived from the mean of all the high and low points on the tidal curve whereas *MSL* is derived from the mean of a much larger number of points taken at hourly intervals along the tidal curve.

215. Mean Low Water

Mean low water (*MLW*) has been generally adopted as the datum for hydrographic surveys along the Atlantic coast of the United States. It is the mean of all low waters as observed over a long period. When the record does not cover a full 19-year period, the mean range for the period of record, derived as indicated in paragraph 212 is corrected for the longitude of the moon's node, and one-half of the corrected range is then subtracted from the mean tide level to obtain the corrected mean low water.

216. Mean Lower Low Water

This datum, (*MLLW*), has been generally adopted for hydrographic surveys along the Pacific coast of the United States, Alaska, Hawaii, and the Philippine Islands. It is the mean of the lower of the two low waters for each day over a long period. For several years of record the mean range and the diurnal low water inequality are corrected for the longitude of the moon's node. The sum of the corrected half-tide range and the corrected diurnal low water inequality is then subtracted from the value of mean tide level to obtain the corrected mean lower low water.

217. Mean Low Water Springs

This datum, (*MLWS*), is used on the Pacific coast of the Panama Canal Zone. It is defined as the mean of the low waters of the spring tides occurring a day or two after new or full moon and is obtained by subtracting one-half of the range of the spring tides from the mean tide level. By observation and analysis, it has been found that the ratio of the spring range to the mean range is fairly constant over rather wide areas. For the Pacific coast of the Canal Zone, this ratio may be taken as 1.26. For this locality therefore, the datum of mean low water springs equals half-tide level minus ½ x 1.26 times the mean range of the tide.

218. Other Datums

Other datums such as mean high water (*MHW*) and mean higher high water (*MHHW*) are sometimes used as a basis for leveling operations. They have been defined in paragraph 211 and are computed as indicated in figure 81. Still other datums have been used in foreign countries.

219. Tide Reducers for Soundings

After the value for the tidal datum has been obtained and all tide stations in an area have been referenced to it by differential leveling, the tide reducers for soundings are obtained by subtracting the gage reading corresponding to the datum from the recorded heights of the tide taken at intervals during the sounding operations. The differences normally will be positive except when the tide falls below the level of the datum. Positive differences are to be subtracted from actual depths obtained by sounding. When entered in the sounding notes of the hydrographic survey, the minus sign is usually omitted for convenience. When the tide falls below the datum, the difference must be added to the depth obtained by sounding. In this case the difference must be prefixed by a plus sign when entered in the sounding record. Water depths normally are measured in integral feet or meters. Even in shoal water, readings are only taken to the nearest half foot (0.15m). It is therefore unnecessary to correct each sounding by the exact stage of the tide, above or below datum, which corresponds to the instant when the particular sounding was taken. Instead, a reducer in integral feet or meters is used for all soundings taken during an applicable period of time. These reducers and time periods can be determined graphically by placing a transparent overlay on the tide-

curve marigram. Horizontal and vertical rulings are drawn on the overlay, the ordinates representing feet on the height scale of the marigram and the abscissas representing hours on the time scale. Properly positioned on the marigram with reference to the datum of the gage, it is possible to read the time period during which each reducer should be applied. These are tabulated as indicated below:

Date	Reducer (feet)	Time period (hours)
2 April 19__	—2	0800–0830
	—3	0830–0930

and the like. Where there is an appreciable difference in time of tide between the gage location and the area in which soundings are being taken, the overlay must be offset horizontally on the marigram to account for this time differential.

220. Approximate Tide Predictions

Engineer operations requiring a knowledge of the time and heights of tide are frequently carried out at points for which predictions are not published in the annual tide tables (para. 193). Approximate predictions for several weeks in advance can be made on the basis of a comparison between a tide record covering a few days at the point in question and the observed or predicted tides during the same period at the nearest reference station. Such a comparison is shown in figure 82. Columns 2, 3, 4, and 5 show the times and heights of high and low water at the reference tide sta-

Year 19	Reference Station A				Secondary Station B				Differences			
	Time of		Height of		Time of		Height of					
Day Month	High Water	Low Water	High Water	Low Water	High Water	Low Water	High Water	Low Water	6-2	7-3	8-4	9-5
1	Hr. Dec. 2	Hr. Dec. 3	Feet 4	Feet 5	Hr. Dec. 6	Hr. Dec. 7	Feet 8	Feet 9	10	11	12	13
1 Sept.	8.00	2.00	7.75	3.05	7.75	2.00	8.07	2.93	-0.25	0.00	+0.32	-0.12
	20.00	14.25	8.67	3.25	19.50	13.75	9.17	3.39	-0.50	-0.50	+0.50	+0.14
2 "	8.50	2.75	8.77	3.60	8.25	3.00	9.17	3.79	-0.25	+0.25	+0.40	+0.19
	20.75	15.00	8.68	3.82	20.25	15.00	9.05	3.84	-0.50	0.00	+0.37	+0.02
3 "	9.50	3.00	8.50	3.17	9.00	3.25	8.74	3.10	-0.50	+0.25	+0.24	-0.07
	21.25	15.50	8.14	3.42	21.00	15.25	8.35	3.27	-0.25	-0.25	+0.21	-0.15
4 "	10.00	3.75	8.48	3.08	9.75	3.75	8.72	2.92	-0.25	0.00	+0.24	-0.16
	22.00	16.00	7.86	3.20	21.75	15.75	8.06	3.05	-0.25	-0.25	+0.20	-0.15
5 "	10.50	4.25	8.48	2.94	10.25	4.25	8.80	2.78	-0.25	0.00	+0.32	-0.16
	23.00	16.75	7.85	3.38	23.00	16.50	8.15	3.41	0.00	-0.25	+0.30	+0.03
6 "	11.00	5.00	8.70	3.25	11.25	4.75	9.00	3.21	+0.25	-0.25	+0.30	-0.04
	23.75	17.75	7.70	3.58	23.50	17.25	8.00	3.50	-0.25	-0.50	+0.30	-0.08
7 "	—	5.75	—	3.67	—	5.75	—	3.61	—	0.00	—	-0.06
	12.00	18.75	8.39	3.54	12.25	18.50	8.62	3.50	+0.25	-0.25	+0.23	-0.04
								Means	-0.21	-0.12	+0.30	-0.05
30 "	7.25	1.25	7.77	2.56	7.04	1.13	8.07	2.51				
	19.25	13.75	7.97	2.54	19.04	13.63	8.27	2.49				

Figure 82. Comparison of simultaneous tidal observations.

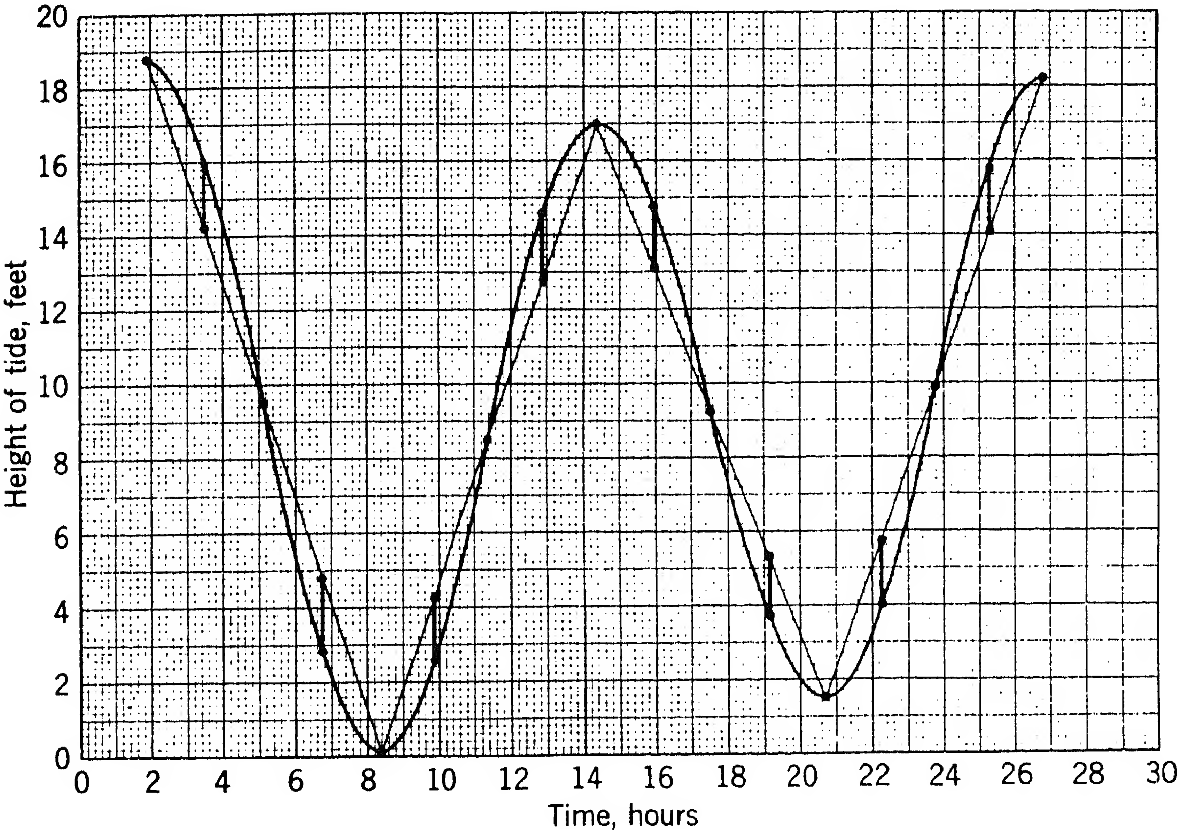


Figure 83. Tide curve constructed by one-quarter, one-tenth rule.

tion of an East Coast port during the first week in September 1951. Columns 6, 7, 8, and 9 show similar values as observed at a secondary station some miles distant. It was desired to predict the times and heights of tide at this secondary station on 30 September 1951 in connection with subaqueous construction operations. Columns 10 and 11 show the differences in time of high and low water and columns 12 and 13 show the differences in the heights of high and low water at the two stations. The means for the week for these four columns are indicated. At the bottom of the sheet are shown the predicted values for 30 September at the reference station as taken from tide tables. The means of the last four columns are applied these values to obtain predicted times and heights of tide for the secondary station on this same date. Results which are satisfactory for many military purposes can be obtained from a short record of this type. Better results are obtained when the record can be kept for an entire lunar month.

221. Construction of Approximate Tide Curve

The annual tide tables (para. 193) or the method described above will furnish predicted values of the times and heights of high and low water at a station. In addition to such data, the actual tide curve for the station is required when soundings are to be taken in the vicinity so that appropriate tide reducers (para. 219) can be applied to measured depths of water.

The *one-quarter, one-tenth rule* can be applied in the construction of a curve which will closely approximate the actual tidal curve. The procedure is as follows:

a. Plot, on cross section paper (fig. 83), the high and low water points for the date desired in the order of their occurrence, measuring time horizontally and height vertically. These are the basic points for the curve.

b. Draw light straight lines connecting the points representing successive high and low waters.

c. Divide each of these straight lines into four equal parts. The halfway point of each line gives another point for the curve.

d. At the one-quarter point adjacent to high water draw a vertical line above the point and at the one-quarter point adjacent to low water draw a vertical line below the point, making the length of these lines equal to one-tenth of the range of tide between adjacent high and low waters. The points marking the ends of these vertical lines furnish additional intermediate points for the tide curve.

e. Draw a smooth curve through the points of high and low waters and the intermediate points, making the curve well rounded near high and low water. This curve will closely approximate the actual tide curve, and the tidal height for any time of the day may be scaled from it. The curve shown (fig. 83) has been plotted from the following data:

15 April 19___, Eastport

High		Low	
Time	Height (feet)	Time	Height (feet)
h m		*h m*	
1 52	18.8	8 23	0.1
14 26	17.0	20 42	1.5
2 48 (16 April)	18.2		

Section III. STREAMFLOW AND STREAM-GAGING STATIONS

222. Need for Streamflow Records

Data concerning the quantity and velocity of flow of streams are necessary in the solution of many engineering problems involving the construction of temporary or permanent river crossings, surface-water supplies, flood-control structures, and works required for inland waterways. In some instances, records covering a short period of time will suffice. Usually, however, records showing the variations in streamflow over a long term of years are most useful or even essential. It is only on the basis of data covering a long period that reasonably accurate predictions of flood flows and of the extent of periods of minimum runoff can be made and the various hydrologic structures can be safely and economically designed.

223. Factors Influencing Streamflow

Streamflow or runoff is a residual, being equal, over a period of time, to the precipitation of the watershed of the stream less the amount of infiltration to underlying groundwater horizons, transpiration, and evaporation. Precipitation may occur in the form of rainfall, snow, or hail, and, to a much lesser extent, as dew or frost. The variations in streamflow depend not only upon the variations in, and the form of, precipitation but upon all the factors which affect and control infiltration, transpiration, and evaporation. Infiltration is dependent upon the perviousness of the soil and underlying rock, the steepness or flatness of the ground surface, the presence or absence of vegetal cover, the extent of frost in the soil, and other factors. Transpiration varies with the extent and type of vegetal cover and with the season of the year as it controls the rate of growth of vegetation. Evaporation is controlled by temperature, wind velocity, relative humidity, barometric pressure, altitude, and many other meteorological, topographic, geographic, and climatological conditions. No simple relationship can be set up between the quantity of precipitation on a watershed and the resulting runoff. It would greatly simplify the design of hydrologic structures if this were true, for precipitation records for long periods are available at a very large number of points, but it is clear that actual records of runoff form a far more satisfactory basis for design and construction.

224. Units of Measurement

The velocity of flow of streams is commonly expressed in feet per second. For certain purposes, velocity is given in miles per hour. Velocity, particularly of tidal currents, may also be expressed in knots (nautical miles per hour). One knot is equivalent to 1.1515 statute miles per hour. The quantity of runoff is normally measured in cubic feet per second, commonly abbreviated *cfs* or *second-feet*. It may be expressed in second-feet per square mile of contributory drainage area. For a given period of time, such as a month, the quantity may be stated in inches, that is, the depth in inches on the drainage area, assuming the total monthly quantity to be spread evenly over the entire watershed. This permits comparison of runoff with precipitation and evaporation or other water losses which are usually given in inches. Thus, if the drainage area above a gaging station totaled 100 square miles and the runoff at the station averaged 50 cubic feet per second throughout the month of June, the monthly runoff in inches would equal—

$$\frac{50\ \frac{\text{cu ft}}{\text{sec}} \times 86400\ \frac{\text{sec}}{\text{day}} \times 30\ \frac{\text{days}}{\text{mo}} \times 12\ \text{in}}{100\ \text{sq mi} \times 5280 \times 5280\ \frac{\text{sq ft}}{\text{sq mi}}} = 0.557\ \frac{\text{in}}{\text{mo}}$$

Runoff may also be expressed in miner's inches, the quantity passing a one-inch square orifice in a unit time under a specified head. It may also be measured, for a definite period of time, in acre-feet.

225. Records of U.S. Geological Survey

The measurement of streamflow and the compilation of records of runoff for streams of the United States and its territorial possessions is a function of the Water Resources Branch of the U.S. Geological Survey acting, in many instances, in cooperation with other federal, state, county, municipal, and private agencies. As the need for such records has become more evident, appropriations for the work have been increased until there are now more than 4,000 points on streams where gagings are made. The records are published annually in about 12 volumes of water supply papers entitled Surface Water Supply of the United States, each volume covering streams of a section of the country. These volumes are available through the Superintendent of Documents, Washington, D. C. There is inevitably a considerable lag between the gathering of the data and the printing of the volume for a particular year. When the latest records for a particular station are desired, the data may be obtained in blueprint form from the U.S. Geological Survey District Engineer shortly after the close of the water year. The water year differs from the calendar year. In the northeastern section of the country it runs from 1 October to 30 September. This permits a better comparison of annual precipitation and runoff than would be possible if the calendar year was used. In the latter case, much precipitation falling as snow in December of one year would not appear as runoff until the spring of the following year.

226. Stream-Gaging Stations

A stream-gaging station is located on a stream at a point at which periodic measurements of velocity or discharge are made and at which daily or continuous records of the stage or height of the water surface above a given datum are obtained. It is a laborious and time-consuming operation to measure the discharge, whereas it is an easy matter to obtain a record of the rise and fall of the water surface by means of a *water-stage recorder*. This instrument is similar in character to the automatic tide gage described in paragraph 201. There are two models of water-stage recorders which are items of military issue. By measuring the discharge on a number of occasions at different stages of the stream it is possible to establish a stage-discharge relationship for the station. This may be shown graphically (fig. 84) as a *station-rating curve* or in tabular form as a *station-rating table*. Once this stage-discharge relationship has been determined, the stage at any time can be read from a gage or from the record of the water stage recorder and the corresponding discharge taken immediately from the record of the water stage recorder and the corresponding discharge taken immedi-

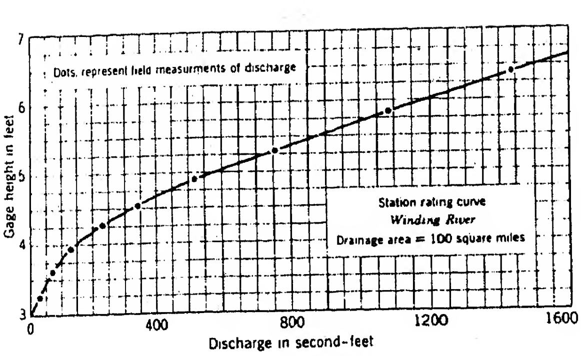


Figure 84. Typical station-rating curve.

ately from the station-rating curve or table. Henceforth, it is only necessary to measure the discharge periodically for assurance that the stage-discharge relationship has remained unchanged or to establish a new relationship when changes in the stream channel render it necessary to do so.

227. Sections at a Stream-Gaging Station

There are three cross sections of importance at a gaging station. These are the *gaging section*, the *measuring section*, and the *control section*. Their characteristics are discussed in paragraphs 228 through 230.

228. Gaging Section

The gaging section is that at which the gages and the water-stage recorder are located to measure the water level of the stream. The recorder and the sensitive gages are located in a gage house built over a well which is connected to the stream by pipes. Other gages may be situated outside. A permanent bench mark should be located near the gage house to permit referencing and checking the gages.

229. Measuring Section

The measuring section is that at which velocity and discharge measurements are made, usually by means of a current meter. It may be at the gaging section of a short distance upstream or downstream from it. Where not located at the gaging section, the same quantity of flow should pass both, that is, no appreciable quantity of surface runoff or flow from a tributary stream should enter the main stream between the two sections.

230. Control Section

The gaging section should be located upstream from a control section which serves to regulate the height of the water surface at the gage. The control may consist of a ledge of rock extending across the stream channel, a gravel bar, a number of large boulders in the channel, a falls or rapids, or a dam or weir constructed for the purpose. The control restricts the flow of the stream at that point and is designed to stabilize the stage-discharge relationship at the gage and to eliminate or minimize backwater effects resulting from downstream conditions. A control should restrict the flow to such an extent that it can be considered as having a sensitive effect at the gage. A small change in discharge should show a measurable change in stage at the gage since it is these gage readings which are being used to determine the discharge from the station-rating curve. Sometimes one control may operate at low water while a second, downstream, becomes effective at higher stages. A control is always desirable but it may be impossible either to locate a natural control or to construct an artificial one below a point where streamflow measurements are desired. In this case, more frequent checks of the stage-discharge relationship will be necessary and it must be recognized that the accuracy of the record will be affected during periods of backwater from tributaries downstream.

231. Typical Gaging Stations

Figure 85 shows a typical stream-gaging station. A concrete weir, so shaped as to provide sensitivity at both high and low stages, forms the control in the foreground. Upstream, the gaging section is indicated by the gage house and staff gage on the bank. The measuring section is just above the gaging section, measurements being made in this case with a current meter lowered from a small car carried on a cable suspended across the stream. Figure 86 shows a rough plan of another gaging station. Here, the control consists of an island and rock reef forming rapids in the stream and constricting the flow to provide sensitivity at the gage upstream which is located on a smooth reach. At high water, current-meter measurements are made from the single span railroad bridge below the control. At low-water stages the width here is so great that the

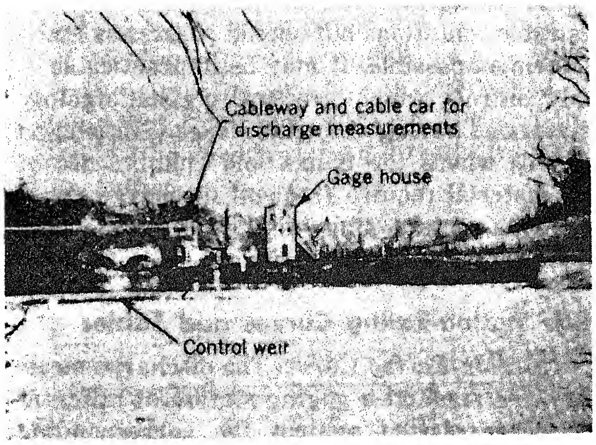


Figure 85. Typical stream-gaging stations.

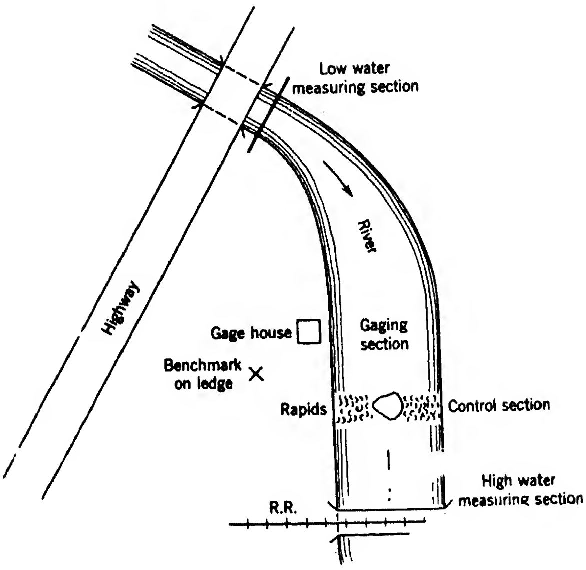


Figure 86. Plan of a stream-gaging station.

velocities are too low for accurate measurement with the current meter, and measurements are made by wading just below the highway bridge where the constricted channel provides higher velocities. The distance between the two bridges is about one-fourth of a mile (0.4 km). No tributaries enter the river between the bridges so that the quantity of flow is substantially the same at the two points at any instant.

232. Criteria Controlling the Location of Gaging Stations

a. A well chosen stream-gaging station site should possess the following characteristics:

(1) A sensitive, permanent control section below the gage with sufficient drop in the stream as it moves through the control section to prevent any backwater effects, originating below the control section, from reaching the gaging section.

(2) A long straight channel extending through the measuring section so that the water filaments will follow straight paths and errors caused by eddies and turbulence will be minimized.

(3) Measuring and gaging sections where the channel is in rock or hard material not subject to scour so as to avoid frequent changes in the stage-discharge relationship.

(4) A station well below rapids or stretches of high-velocity flow so that sediment carried by the stream in those reaches will be deposited above the station. Sediment deposited at the station will change the area and alter the stage-discharge relationship.

(5) A relatively uniform channel shape at the measuring section (fig. 87), rather than an irregularly shaped section (fig. 88). The station-rating curve for the latter would show a sharp break at the point where a small increase in depth results in a considerable increase in cross section area. Additional discharge measurements are required to correctly define such a curve.

(6) A gaging section with high banks so that the water-stage recorder will not be inoperative during flood flows.

(7) A section unaffected by abundant

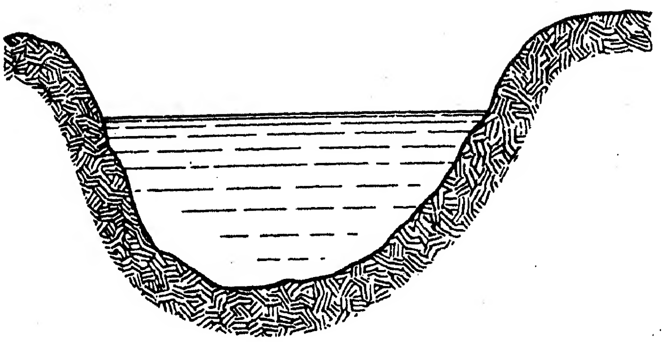


Figure 87. Uniformly shaped measuring section.

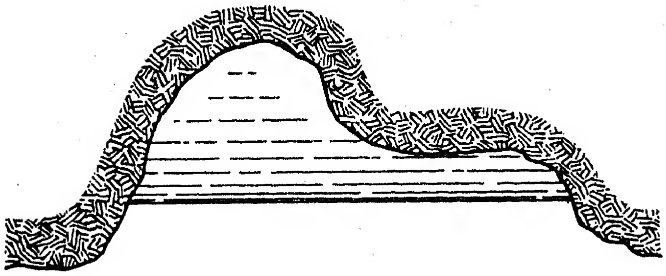


Figure 88. Irregularly shaped measuring section.

growth of aquatic vegetation during the summer. Such vegetation reduces velocity and hence increases the stage for a given discharge.

(8) A station where a permanent bench mark can be established near the gage.

(9) A station accessible to roads when it is necessary to visit the station to measure the discharge, read the gages, wind the clockwork, or change the record paper of the water-stage recorder.

(10) A measuring section where satisfactory velocity and discharge measurements may be made by wading or from a cableway, bridge, or boat. A single-span bridge is preferable to one having intermediate piers because of eddies and transverse velocity current caused by the obstructing piers.

(11) A section at a point on the stream where records are likely to be of optimum use.

b. A station site will seldom meet all of these desirable conditions but should possess as mar of them as possible. It may be impracticable find controls which are entirely satisfactory on streams having very flat slopes. Stations on streams where the banks and channel are of soft material require frequent discharge measurements as the stage-discharge relationship is subject to continuous change.

233. Station-Rating Curves and Tables

a. Station-Rating Curve. The discharge measurements made at a gaging station at different times are plotted against the corresponding gage heights to form a station-rating curve. This plot may be on ordinary Cartesian coordinate paper, as shown in figure 84. It is preferable to adjust the gage readings to heights above zero flow and to plot the values on logarithmic paper. The adjusted curve will then plot as a straight line. The advantage of the logarithmic plot is the ease of extrapolation to determine the discharge during flood stages. The height to which the water rises is readily obtained from marks on trees even if the water-stage recorder is rendered inoperative by the floodwaters. On the other hand, actual meas urements of discharge are seldom possible a these high stages. Figure 89 shows a typical station-rating curve plotted in this manner. The zero of the gage is initially set below the elevation of zero flow to assure a positive reading at all times. The gage height for zero flow may be determined from observations or may be established by trial. In the latter case, different values are assumed for the gage height of zero flow and station-rating curves are

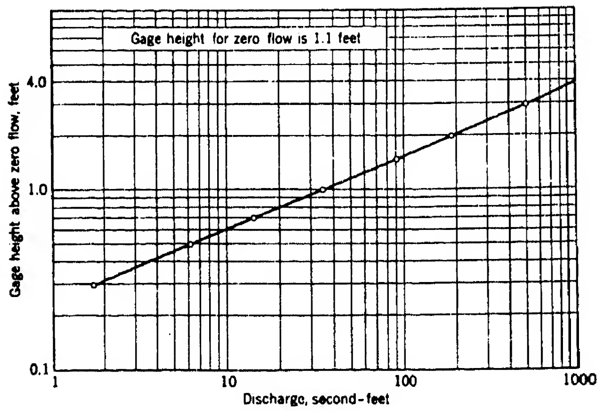


Figure 89. Portion of logarithmic rating curve.

plotted on logarithmic paper until a value is found which produces a straight-line plot. Once the gage height for zero flow is determined, it is subtracted from each gage reading and the corresponding discharge read directly from the rating curve.

b. Station-Rating Table. From the station-rating curve a rating table can be prepared which gives the discharge for every 0.1 or 0.01 foot of gage height or of height above zero flow. Table VI shows a portion of such a rating table.

Table VI. Portion of Typical Rating Table

Rating Table for Clear River

Gage height	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	Difference
Feet	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*
1.0	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	
.1	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	
.2	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	
.3	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	
.4	1.7	1.9	2.1	2.2	2.4	2.6	2.8	2.9	3.1	3.3	
.5	3.5	3.7	3.9	4.1	4.3	4.6	4.9	5.2	5.5	5.8	
.6	6.1	6.4	6.7	7.0	7.3	7.7	8.1	8.5	8.8	9.2	Curve
.7	9.6	10.0	10.4	10.8	11.2	11.6	12.1	12.6	13.1	13.6	4.5
.8	14.1	14.7	15.3	15.8	16.4	17.0	17.6	18.2	18.7	19.3	5.8
.9	19.9	21	21	22	23	23	24	25	26	26	7.1
2.0	27	28	29	30	31	32	32	33	34	35	9
.1	36	37	38	39	40	41	42	43	44	45	10
.2	46	47	48	50	51	52	53	54	56	57	12
.3	58	59	61	62	64	65	66	68	69	71	14
.4	72	74	75	77	79	80	82	84	86	87	17
.5	89	91	93	95	97	99	101	103	105	107	20
.6	109	111	114	116	119	121	123	126	128	131	24
.7	133	136	138	141	144	146	149	152	155	157	27
.8	160	163	165	168	170	173	176	178	181	183	26
.9	186	188	191	194	196	198	201	204	206	208	25
3.0	211	213	216	218	220	222	225	227	229	232	23
.1	234	236	239	241	244	246	248	251	253	256	24
.2	258	260	263	266	268	270	273	276	278	280	25
.3	283	286	288	291	293	296	299	301	304	306	26
.4	309	312	315	317	320	323	326	329	331	334	28
.5	337	340	343	346	349	352	354	357	360	363	29
.6	366	369	372	375	378	381	384	387	390	393	30
.7	396	399	402	405	408	412	415	418	421	424	31
.8	427	430	434	437	440	444	447	450	453	457	33
.9	460	463	467	470	474	477	480	484	487	491	34

234. Installation and Operation of Gaging-Station Equipment

The installation and operation of equipment at a stream-gaging station is covered in paragraphs 235 through 246, following a description of the individual instruments and items of equipment.

235. Stream-Gaging Instruments

The instruments used for stream-gaging operations may be classified in three general groups: instruments for measuring differences in water levels (including gages of various types); instruments for measuring velocity (including floats and current meters); and instruments for measuring depth of water.

236. Gages

Gages of several types are used to measure the water level at a point in a stream at a given instant of time, or to record the changes in water level with the passage of time. Rough measurements are read from gages secured to the river bank, to a bridge pier, or to some other supporting structure in the stream. Greater accuracy is assured when the gage measures the level in a stilling box or well which is connected to the stream by a pipe or pipes. No attempt is made to describe all of the varieties of gages in this manual. Those described are types which can be readily constructed or improvised or types which are likely to be requisitioned for engineer operations.

237. The Staff Gage

The staff gage used in stream-gaging work is essentially the same as that used for measurements of tidal heights (para. 199). The staff gage is commonly mounted vertically on the wall of a stilling box or on a bridge pier or other support in such a position that the stage of the stream is read directly from the intersection of the water surface with the graduated staff. Painted graduations will suffice for short-term operations. More permanent gages have graduations formed on cast iron or porcelain-enameled metal strips (fig. 90). In some instances, supporting structures for a vertical gage are not available and it may be more convenient to use an inclined gage, the graduations being fastened to a heavy timber securely anchored and resting on the sloping bank of the stream. The slope distance between graduations representing a certain vertical rise or fall in the stream is dependent upon the slope of the bank so that each such gage must be designed for a particular location.

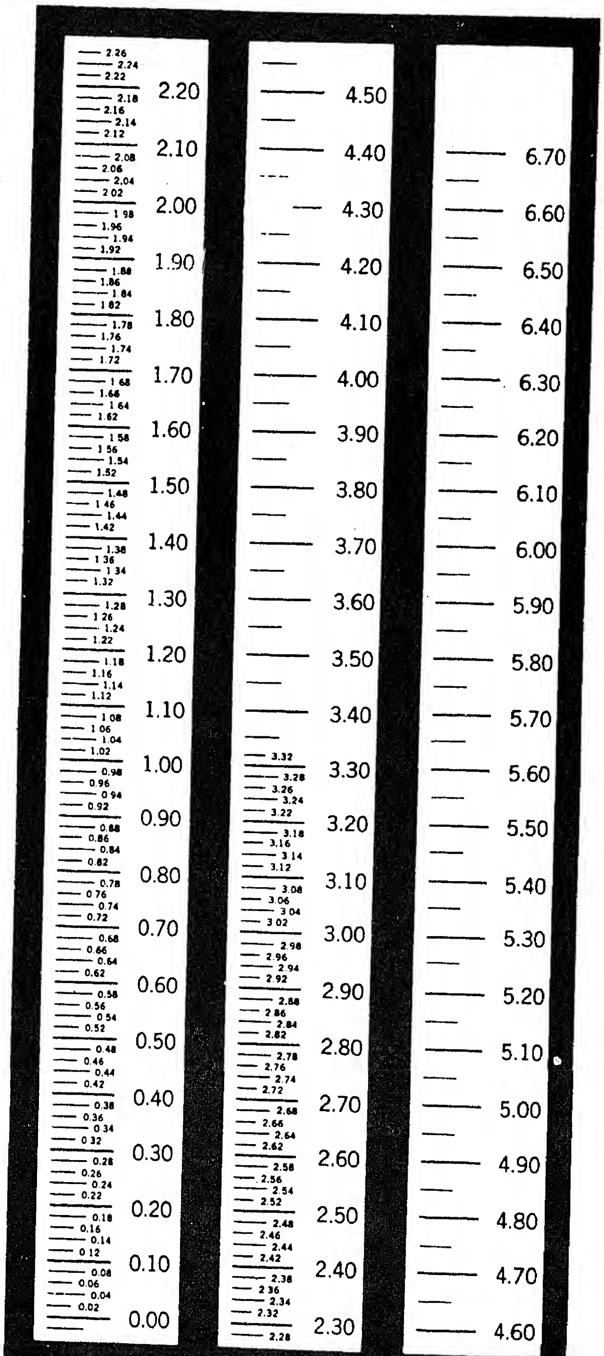


Figure 90. Metal strip graduations for staff gage.

238. The Chain Gage

The chain gage is commonly mounted on a bridge or other structure overhanging the

stream. It is so located and so operated that it is protected from ice and floating drift which might damage or destroy a staff gage. This gage consists of a board which projects horizontally from a gage box. The board carries a painted or enameled metal strip scale. The gage box contains a pulley over which runs a heavy sash chain carrying a weight at one end and having an index marker attached near the other. The box has a hole beneath the pulley for the weight to pass through. As the weight is lowered to the water surface, the chain passes over the pulley and the index marker moves along the graduated scale. The gage box and scale are so arranged that when the bottom of the weight contacts the water surface, the gage height is read on the scale opposite the index marker. An adjustment is provided at the top of the weight to permit maintenance of a constant distance from the bottom of the weight to the index marker, regardless of chain wear.

239. Plumb Bob and Tape

Water levels can be measured accurately by using a plumb bob and tape in a simple adaptation of the principle of the chain gage. A plumb bob is attached to a steel tape which is passed over a pulley or rounded surface and referred to a fixed horizontal scale. A definite graduation mark on the tape is used as an index in reading the gage height of the scale when the point of the plumb bob has been lowered to the water surface.

240. Recording Gage

A recording river gage or water-stage recorder is similar in principle to the automatic tide gage (para. 201). A number of types are manufactured commercially. These differ somewhat in construction and each contains special features which adapt it to use under particular conditions. One type is shown in figure 91. All of these instruments are designed to produce a graphic or printed record of the changes in water level with respect to time. They are provided with both a gage-height element and a time element which, operating together, produce a continuous record of the rise and fall of the stream. The time element consists of clockwork, electrically-, spring-, or weight-driven, with the necessary accessories to reproduce the time record on a chart. The gage-height element is actuated by either a float or a pressure device and has auxiliary equipment required to transfer the variations in stage to the chart. The records obtained from water-stage recorders are superior to and have many advantages over gage records obtained from periodic observations. Gages of the other types are used largely for temporary installations, or to set and check the recording gages.

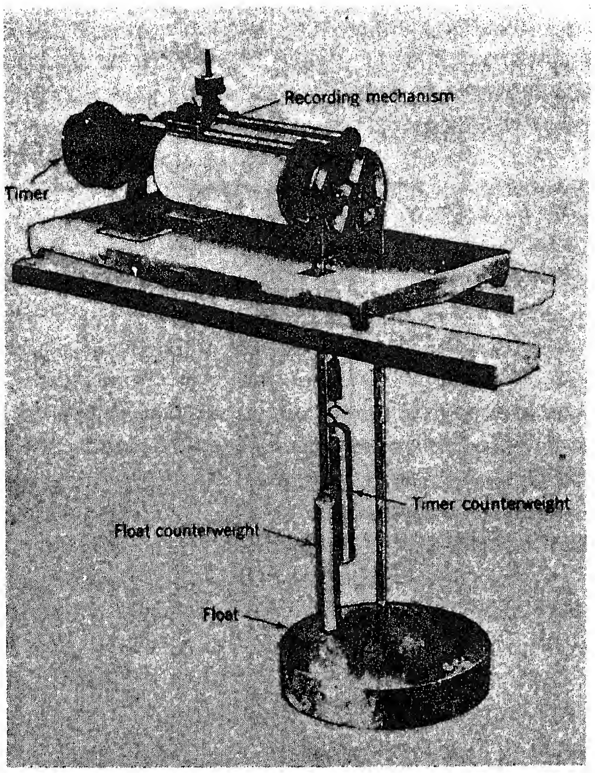


Figure 91. Water stage recorder.

241. The Float Gage

The float gage (fig. 92), similar in principle to the instrument of the same name used in tidal observations is often installed in the well of a water-stage recorder to provide a check on the readings of that instrument. The gage consists of a float connected to a counterweight by means of a stainless-steel graduated tape passing over a pulley. The forward portion of the pulley standard is flanged, the top edge of the flange providing a reference mark for reading the tape. Means for making adjustments in the length and setting of the tape are provided.

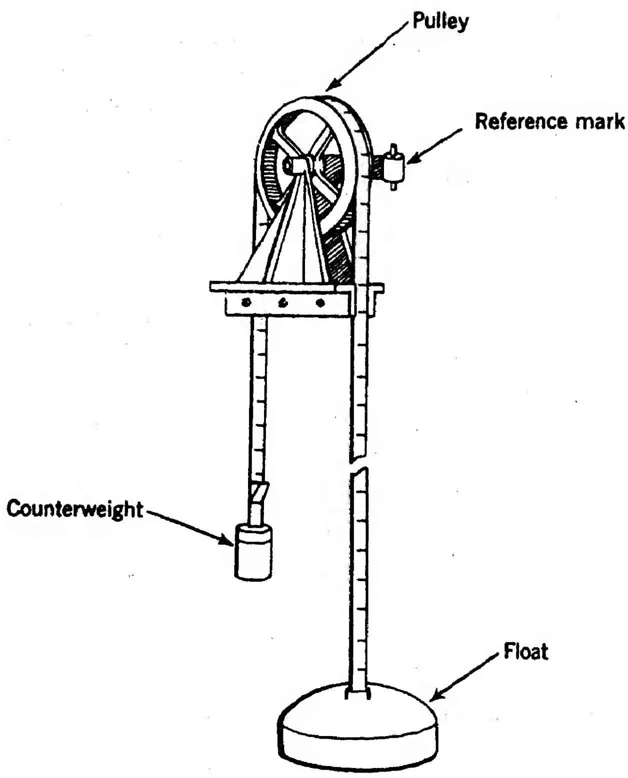


Figure 92. Float gage.

242. The Hook Gage

The hook gage is used for precise measurement of the head of water flowing over a weir, generally for refined work of short duration. The gage is installed in a stilling box at any convenient point near the weir, the water being conveyed to the box by a pipe. The water in the box being at rest, its surface indicates the precise water level above the weir. To measure the depth of water flowing over the weir, the level of the crest is determined with a leveling instrument, and this elevation is transferred to a mark on the inside of the gage box. The gage scale is set to read zero, and the gage is fastened to the inside of the box by means of two screws so that the point of the hook is at the same elevation as the mark. The point of the hook will now be under water and level with the crest of the weir. The depth of water flowing over the weir is the distance from the point of the hook in this position to the exact surface of the water. To read the gage, the hook is raised until it pierces the surfaces; it is then clamped in position, and by means of the slow motion screw its height is adjusted until the water surface shows no distortion. This position, which gives the exact elevation of the water surface, is then read on the vernier to thousandths of feet. An advantage of this gage is that after the zero positions have been found it can be carried from one weir to another, and thus duplication of installations may be avoided.

243. Instruments for Measuring Current Velocity

Current velocities are commonly determined either by the use of floats or by the use of a current meter. The three common types of floats used in measuring stream velocity are surface, subsurface, and rod floats.

a. Surface Floats. A surface float may be specially designed or may be improvised from partially filled corked bottles, selected pieces of driftwood, or ice cakes floating with the current. Since a surface float moves, theoretically at least, with the same velocity as the surface water, it does not measure the mean velocity in the vertical longitudinal section traversed. A coefficient must be applied to observed surface velocity to determine the mean velocity in the section. This coefficient is about 0.8 but is quite variable so that the results obtained from the use of surface floats are approximate at best. Furthermore, the movement of surface floats is likely to be greatly affected by wind and cross currents. Floating ice cakes or logs furnish more reliable results than small surface floats since they are largely submerged and are less affected by winds and transverse currents.

b. Subsurface Floats. A subsurface float (fig. 93) consists of a hollow cylinder, with axis vertical, slightly heavier than water, held at a constant depth by the buoyant effect of an indicating surface float to which it is connected by a line of adjustable length. In a canal or channel having a relatively uniform depth, a close approximation to the mean velocity is obtained by placing the submerged float at six-tenths of the mean depth. This corresponds to the thread of mean velocity. The values obtained will be somewhat high because of the pull exerted by the indicating float and connecting line.

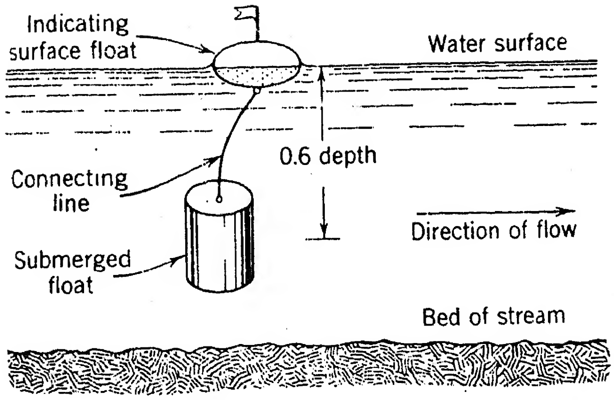


Figure 93. Subsurface float.

c. *Rod Float*. The rod float is usually a cylindrical tube of tin, copper, or brass, 1 or 2 inches (2.5 or 5 cm) in diameter. The tube is sealed at the bottom and is weighted with shot until it will float in an upright position with 2 to 6 inches (5 to 15 cm) projected above the surface of the water. A short section of bamboo fishing rod weighted with mercury is also used; this can be made to float with but ½ inch (12.5 mm) showing above water. The length of the rod should be adjusted to just clear obstructions in the stream bed.

244. Current Meter

A current meter is an instrument used for measuring, indirectly, the velocity of flow in a stream. A major part of the meter consists of a set of cups or vanes which is rotated about a vertical or horizontal axis by the pressure of the flowing water. The rate of rotation bears a direct relationship to the velocity of the water. The instrument is calibrated by moving it, at known velocities, through still water and determining the number of revolutions of the rotor in a given time at various speeds. From these data a calibration curve or table is prepared (table VII). After calibration, if the meter is held at a point in a stream and the number of revolutions of the cups or vanes in a certain interval of time is counted, the velocity of the stream at that point can be determined. By placing the meter at properly selected points in the stream, values can be obtained which will yield the mean velocity in the section. The product of the mean velocity and the area of the section gives the discharge. A current meter of the vertical axis, cup type known as the Price meter has been adopted for use by the U.S. Geological Survey. It is more widely used in this country than any other meter. It has been found most satisfactory for the following reasons:

a. Vertical-axis meters generally operate at low velocities better than horizontal-axis meters and equally well at high velocities. The Price meter is capable of accurately measuring velocities ranging from 0.1 to over 15 feet per second (0.03 to 4.5 m per sec.).

b. The bearings of this meter are placed in air pockets which largely eliminate the entrance of silt to the bearings. Equally satisfactory methods of protecting the bearings of horizontal-axis meters have not been developed.

c. Meter cups which become dented or bent may be repaired in the field without seriously altering the calibration of the instrument, whereas damage to the vanes of a horizontal-axis meter seriously affects its rate of rotation and such damage is difficult to repair in the field.

d. The rotor of the Price meter turns relatively slowly and is adapted to measurement of the full range of velocities normally encountered. A horizontal-axis meter which has vanes pitched at the proper angle for low velocities rotates too rapidly in high-velocity streams and several different rotors varying in vane pitch may be required.

245. Models of the Price Current Meter

a. *Type A Meter*. The parts of the type A Price meter, which is adapted to either wading-rod or cable support and which is the type most widely used, are indicated in figure 94. A complete description of all parts of this instrument, its assembly, adjustment, and care, is given in U.S. Geological Survey Water-Supply Paper 888, Stream-Gaging Procedure. Figure 95 shows two assemblies of the meter for lowering from a bridge or cableway. The assembly on the left shows a length of insulated wire cable inserted between the meter and the rubber-covered suspension cable. This is used to reduce cable friction in deep, high-velocity streams. The usual connection for low-velocity stream-flow measurements is shown

Table VII. Rating Table for Current Meter

This table applies when measurements are made with meter suspended by cable. When measurements are made with meter suspended by rod, reduce the tabular velocities by 2 percent.

Velocity in feet per second

Time (sec.)	Revolutions															
	1	2	3	5	10	20	30	40	50	60	70	80	90	100	150	200
40	0.09	0.15	0.21	0.31	0.58	1.13	1.68	2.23	2.78	3.34	3.90	4.45	5.01	5.56	8.34	11.12
41	.09	.15	.21	.30	.57	1.10	1.64	2.18	2.71	3.26	3.81	4.34	4.89	5.43	8.14	10.85
42	.09	.14	.20	.30	.56	1.07	1.60	2.13	2.65	3.18	3.72	4.24	4.77	5.30	7.95	10.59
43	.09	.14	.20	.29	.54	1.05	1.56	2.08	2.59	3.11	3.63	4.14	4.66	5.18	7.77	10.34
44	.09	.14	.19	.28	.53	1.03	1.53	2.03	2.53	3.04	3.55	4.04	4.55	5.06	7.59	10.10
45	.09	.14	.19	.28	.52	1.01	1.50	1.99	2.48	2.97	3.47	3.95	4.45	4.95	7.42	9.87
46	.09	.14	.19	.28	.51	.99	1.47	1.95	2.43	2.90	3.39	3.87	4.35	4.84	7.26	9.65
47	.08	.14	.18	.27	.50	.97	1.44	1.91	2.38	2.84	3.32	3.79	4.26	4.74	7.11	9.45
48	.08	.14	.18	.26	.49	.95	1.41	1.87	2.33	2.78	3.25	3.71	4.17	4.64	6.96	9.25
49	.08	.13	.18	.26	.48	.93	1.38	1.83	2.28	2.72	3.18	3.63	4.09	4.54	6.81	9.06
50	.08	.13	.17	.26	.47	.91	1.35	1.79	2.23	2.67	3.12	3.56	4.01	4.45	6.67	8.89
51	-----	.13	.17	.25	.46	.90	1.32	1.75	2.19	2.62	3.06	3.49	3.93	4.36	6.54	8.72
52	-----	.13	.17	.25	.46	.88	1.29	1.72	2.15	2.57	3.00	3.42	3.85	4.28	6.42	8.56
53	-----	.13	.16	.24	.45	.86	1.27	1.69	2.11	2.52	2.94	3.36	3.78	4.20	6.30	8.40
54	-----	.13	.16	.24	.44	.85	1.25	1.66	2.07	2.47	2.88	3.30	3.71	4.12	6.18	8.24
55	-----	.13	.16	.24	.43	.83	1.23	1.63	2.03	2.43	2.83	3.24	3.64	4.05	6.07	8.09
56	-----	.12	.16	.23	.43	.82	1.21	1.60	1.99	2.39	2.78	3.18	3.58	3.98	5.96	[illegible]
57	-----	.12	.16	.23	.42	.80	1.19	1.57	1.96	2.35	2.73	3.12	3.52	3.91	5.86	[illegible]
58	-----	.12	.15	.22	.41	.79	1.17	1.54	1.93	2.31	2.68	3.07	3.46	3.84	[illegible]	[illegible]
59	-----	.12	.15	.22	.41	.78	1.15	1.51	1.90	2.27	2.63	3.02	3.40	3.77	5.66	[illegible]
60	-----	.12	.15	.22	.40	.77	1.13	1.48	1.87	2.23	2.59	2.97	3.34	3.71	5.56	7.42
61	-----	.12	.15	.22	.39	.75	1.11	1.46	1.84	2.19	2.55	2.92	3.29	3.65	[illegible]	[illegible]
62	-----	.11	.15	.21	.39	.74	1.09	1.44	1.81	2.16	2.51	2.87	3.24	3.59	5.38	[illegible]
63	-----	.11	.14	.21	.38	.73	1.07	1.42	1.78	2.13	2.47	2.82	3.19	3.53	5.30	[illegible]
64	-----	.11	.14	.21	.38	.72	1.05	1.40	1.75	2.10	2.43	2.77	3.14	3.48	5.22	6.96
65	-----	.11	.14	.20	.37	.71	1.03	1.38	1.72	2.07	2.39	2.73	3.09	3.43	5.14	6.85
66	-----	.11	.14	.20	.37	.70	1.02	1.36	1.69	2.04	2.35	2.69	3.04	3.38	5.06	6.75
67	-----	.11	.14	.20	.36	.69	1.01	1.34	1.66	2.01	2.32	2.65	2.99	8.33	4.98	6.65
68	-----	.11	.14	.20	.36	.68	1.00	1.32	1.64	1.98	2.29	2.61	2.95	3.28	4.91	6.55
69	-----	.11	.13	.19	.35	.67	.99	1.30	1.62	1.95	2.26	2.57	2.91	3.23	4.84	6.45
70	-----	.11	.13	.19	.35	.66	.98	1.28	1.60	1.92	2.23	2.53	2.87	3.18	4.77	6.36

on the right. The streamlined lead weight on the bottom of the weight hanger is designed to keep the cable vertical. Leads of various weights are used, depending upon the velocity of the stream. A 15-pound (6.8 kg) weight is most common. With the heavier weights the cable is wound from a brake-equipped reel carried by a small portable crane (fig. 96) instead of being lowered, held, and raised by hand. The insulated cable not only supports the meter and weight but provides for the electrical connections to the battery and earphone. As the cup assembly rotates, the electrical circuit is completed and broken every revolution when the wire from the cable is connected to the single-contact binding post (4, fig. 94). A buzz sounded in the earphone when the circuit is completed and the number of revolutions of the rotor in a given time is counted in this way. With the connection on the penta-contact binding post (5, fig. 94) the circuit is completed every fifth revolution. This provides for ease in counting when the stream has a velocity in excess of 6 feet per second

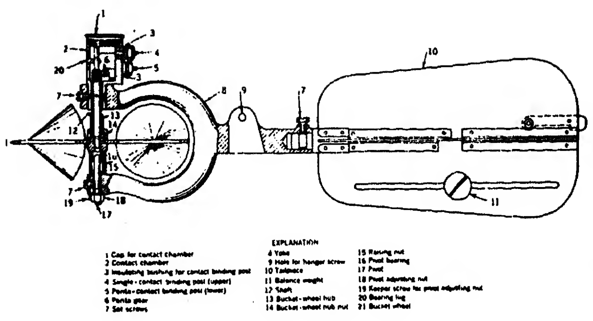


Figure 94. Assembly diagram of type A Price current meter.

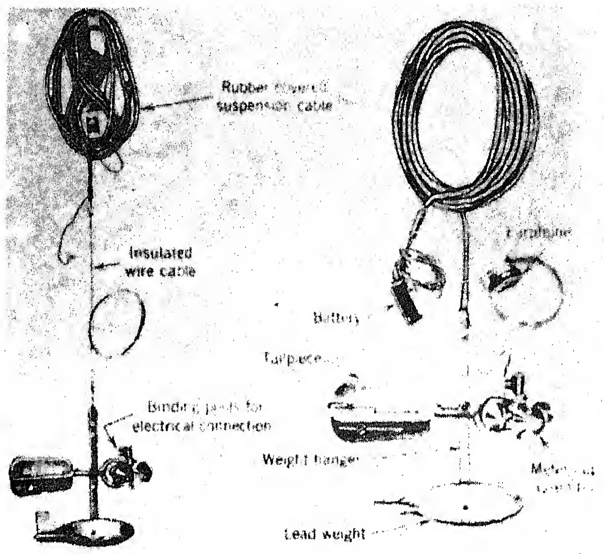


Figure 95. Cable support assemblies of current meter.

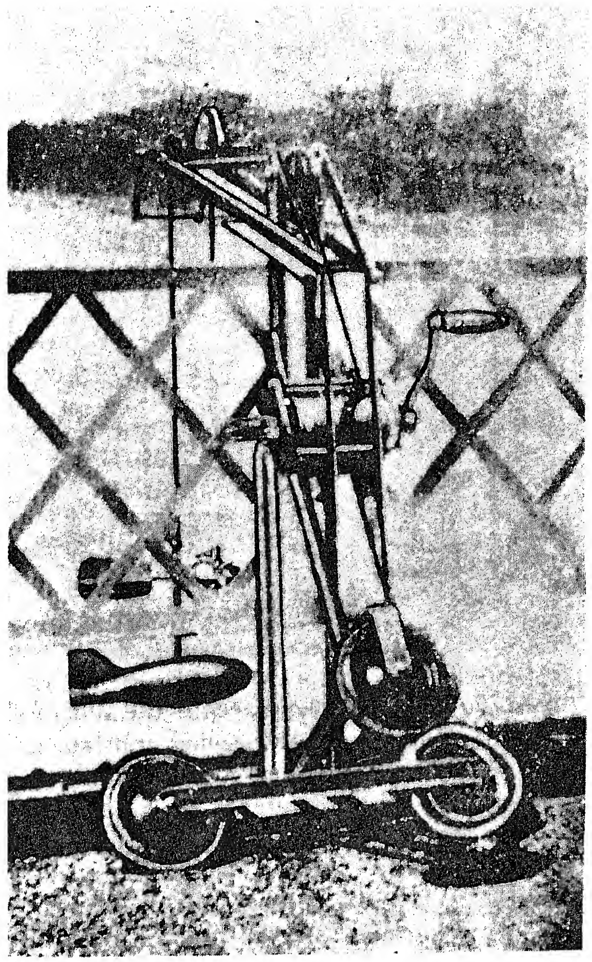


Figure 96. Crane for supporting current meter.

(1.8 m/sec). Figure 97 shows two assemblies of the meter on wading rods for shallow water measurements. The view on the left shows the meter in fixed position on the bottom of the rod to permit readings in very shallow water. The view on the right shows the usual assembly of the meter on the rod above a foot plate. With this assembly, the meter can be clamped on the rod at any desired elevation above the streambed. Figure 98 shows a typical wading-rod measurement. Figure 99 shows a meter suspended from a cableway.

b. Pygmy Type Meter. This meter (fig. 100) has a small cup type bucket wheel, 2 inches (5 cm) in diameter, two-fifths the size of the rotor on the type A meter. It was designed for low-velocity measurements in shallow streams where the type A meter fails to perform with high accuracy. No tailpiece is necessary for these low velocities and the meter is used with wading rods only.

246. Depth-Measuring Equipment for Stream-Gaging

The computation of the discharge at a stream-gaging station involves not only the determination of the mean velocity of flow but the cross section area of the channel as well, the discharge being the product of these two terms. The area is found by measuring the depth at intervals across the stream as indicated in figure 101. Measurements are taken close enough together so that the bottom slope may be assumed as a straight line between depth measurements without introducing ap-

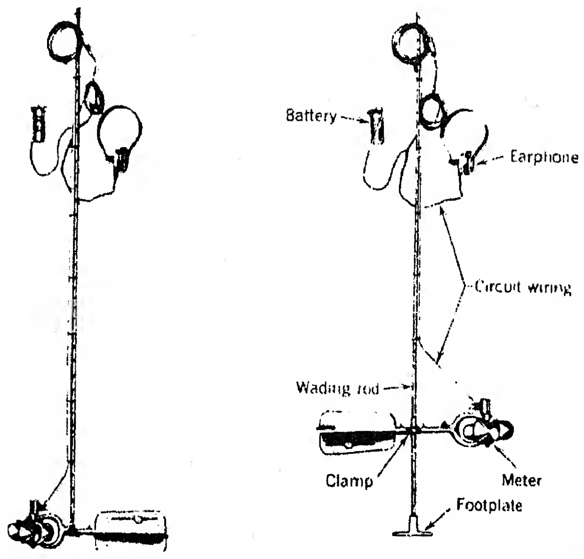


Figure 97. Wading-rod assemblies of current meter.

preciable error. The total area is the sum of the areas of a number of trapezoids and triangles. The distances between soundings may be marked for a tape stretched on a bridge rail or the tape or tag-line may be stretched across the stream where measurements are made by wading. When the current meter is used for velocity determination, the wading rods or the cable and lead weight supplied with the instrument normally are used for the depth measurements. The wading rods are graduated in feet and tenths of feet. A footplate is screwed to the lower end of the assembled rods and is lowered to the streambed, the depth being read at the point where the rod intersects the water surface. When the meter is to be suspended from a bridge or cableway, the lead weight is lowered until it just touches the water surface. The end of a metallic tape is held at the point where the cable passes over the bridge rail or side of the cable car.

247. Gaging-Station Installation and Operation

a. Installation. Where a satisfactory location for a stream-gaging station has been selected (para. 232), the necessary equipment must be installed. Assuming that the station is to be provided with a water-stage recorder, a float well, about 4 feet (1.2 m) square, is dug in

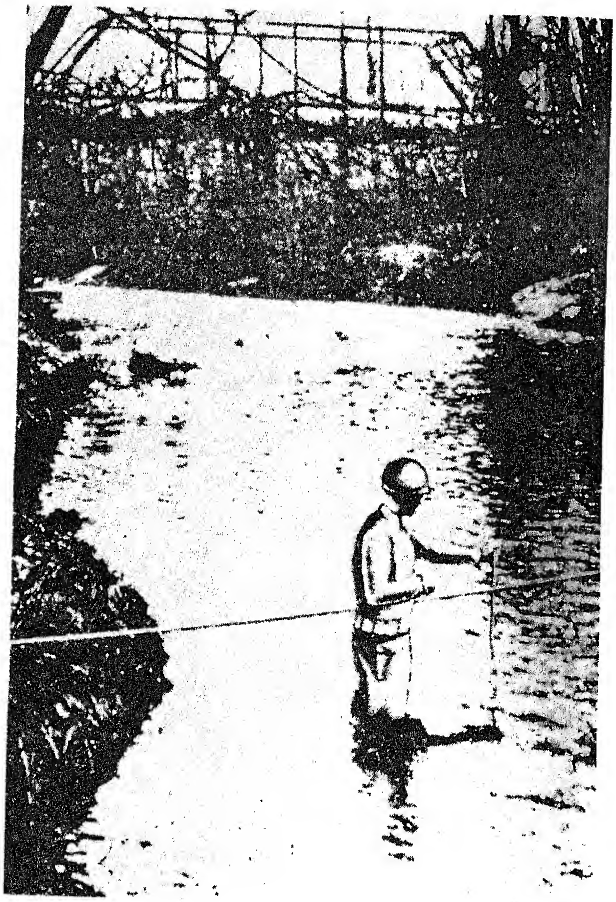


Figure 98. Wading-rod measurement with current meter.

the bank at the gaging section. This well is usually concrete-lined, with metal ladder rungs on one wall provided for access. The well is connected to the stream by intake pipes located below low water level. Two pipes should be used to provide assurance of correct gage readings should one intake become clogged with silt or debris. A wood or concrete gage house, about 5 or 6 feet (1½–2 m) square, is built above the well. A trapdoor is placed in the floor of the house for access to the well. The recorder is mounted on a bench at a convenient height for reading. The recorder should be located above flood stages and, if weight driven, should have sufficient space beneath it for the full movement of the weights. The float well contains the float for the water-stage recorder and an independent gage, usually of the float type, for checking the readings of the recorder. An in-

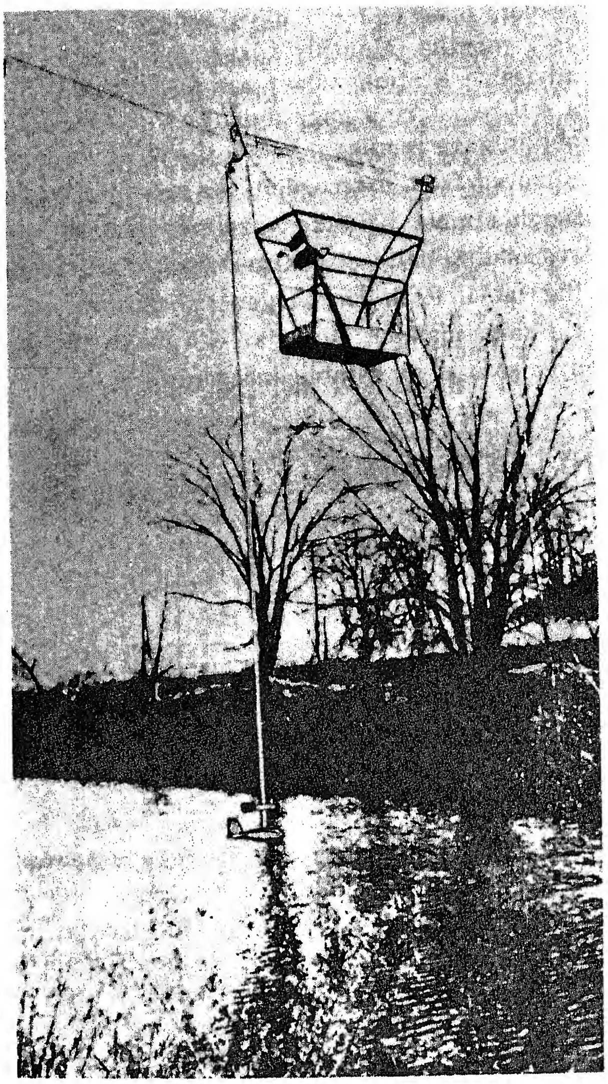


Figure 99. Current meter suspended from cableway.

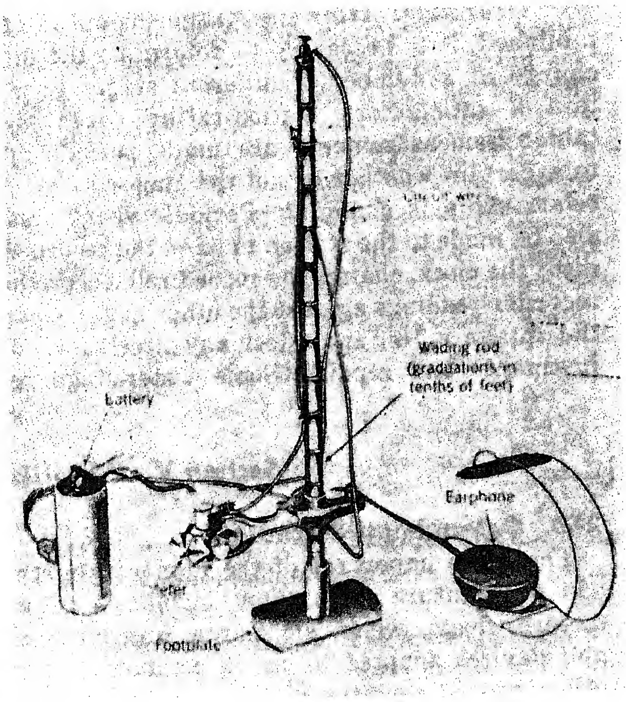


Figure 100. Pygmy current-meter assembly.

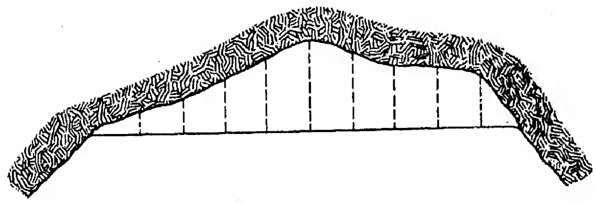


Figure 101. Area of stream cross section as determined by width and depth measurements.

clined or vertical staff gage is erected outside the gage house. This permits stage readings without entering the house and is most helpful in detecting clogging of the intake pipes which would be indicated by a difference in readings on the inside and outside gages. A permanent bench mark must be established nearby, preferably so located that a check on the gages can be obtained from a single setup of the level. If the measuring section is too deep for wading and no bridge is available, a cable is suspended between A-frames on the banks for cable-car operation. Two parallel cables are sometimes used and a wooden footbridge suspended from them. Occasionally measurements are made from a boat but greater accuracy results from the use of the other methods. Where no natural control is present an artificial control normally is constructed. This is usually a concrete weir with a parabolic crest. This will confine the flow, at low stages, to the central portion of the weir. As a result, the gage will show a measurable change in stage for a small change in discharge. This would not be true if the flow were spread over the entire width of a horizontal weir. An installation as described above is desirable when continuous records of streamflow are to be obtained over a long period. When there is a military requirement for records covering a few days or weeks, the only necessary installation will be a staff gage at the point where current meter measurements are made.

b. Operation. After the station has been established, it is necessary to determine the discharge at a number of different stages to obtain a satisfactory station-rating curve and table. Remeasurements are made periodically to ascertain whether or not the stage-discharge relationship is changing. Periodic visits must also be made to the station to wind the recorder, reset the clock, change the record roll, check the recorder readings against the other gages, clean the intake pipes, and effect any necessary adjustments and repairs. Some water-stage recorders operate for 7 days, others for as long as 2 months. Monthly inspections are usually advisable. Stations which are not equipped with a self-registering gage must be visited daily to obtain gage readings. When the chart has been removed from a recorder, the mean daily gage heights must be taken from it and the corresponding daily discharges read from the rating table. Peak discharges are determined in the same manner. One man can make all measurements at an established gaging station.

Section V. MEASUREMENT OF STREAMFLOW

248. Determining Discharge

The more important of the numerous methods to measure streamflow include: current meters, floats, weirs, slope-area, and methods and devices adapted to use in conduits and to other special conditions.

249. Velocity-Area Measurements by Current Meter

The current meter is adapted to velocity measurements of streams under a wide range of conditions. These velocity measurements, combined with determinations of area at the measuring section, are used to compute the discharge of the stream. The meter is designed to measure the velocity at the point in the stream at which the instrument is held. By holding the meter at properly selected points in each vertical where the depth of the stream has been measured (fig. 102), the mean velocity in that vertical can be computed (para. 250). The mean velocity in each of the several verticals across the width of the channel is determined in the same manner. Proceeding from this point, two methods have been used to compute the discharge.

a. Mean-Section Method. The mean velocity in each section or element of cross section area as A (fig. 102) is considered to be the average of the mean velocities in the two verticals which bound it. For example, assume that the mean velocity in the vertical marked 4 (fig. 102) is found to be 1.93 feet per second, that the depth at this point is 3.60 feet, that the mean velocity in the vertical marked 5 is found to be 1.87 feet per second, that the depth at 5 is 3.46 feet, and that the horizontal distance between all verticals is 5.0 feet. The mean velocity in section A is then $\frac{(1.93 + 1.87)}{2} =$ 1.90 feet per second. The area of section A is $\frac{5(3.60 + 3.46)}{2} = 17.6$ square feet. The discharge through section A is then the product of the mean velocity and the area of the section or $1.90 \times 17.6 = 33.4$ cubic feet per second. The discharges for other sections across the width of the channel are computed in a similar manner. The sum of all the partial discharges gives the discharge of the stream. Verticals should be so spaced that not over 5 or 10 percent of the total discharge passes through any one section. This method has been used for many years, is described in surveying texts, and may be preferred by those engineers who are familiar, through long experience, with its application.

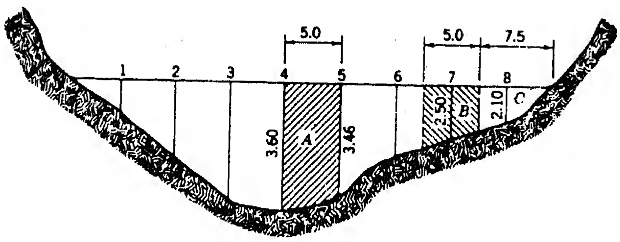


Figure 102. Verticals and sections of a measuring station.

b. Midsection Method. Recently the U.S. Geological Survey has adopted the midsection

method which reduces the amount of computation involved without significant loss of accuracy. Each element of cross section area is considered to be bounded by imaginary verticals midway between those verticals at which measurements are taken. Using this procedure, the area of section B (fig. 102) is taken as the product of the distance between dotted verticals (5.0 ft) and the depth at vertical number 7 (2.50 ft) or 12.5 square feet. The mean velocity in section B is taken as the mean velocity in vertical number 7. Assume that this is found to be 1.22 feet per second. The discharge through section B is then $12.5 \times 1.22 = 15.2$ cubic feet per second. The discharges for other trapezoidal-shaped sections across the width of the stream are computed in the same manner. For the end section C (fig. 102) the only depth and velocity measurements have been made in vertical 8. This whole section is essentially a triangle of base 7.5 feet and altitude equal to the depth at $8 \times \frac{3}{2}$. If this depth is 2.10 feet and the mean velocity in vertical 8 is 1.02 feet per second, the discharge through C is this velocity times the area of the triangle or $1.02 \times \frac{1}{2} \times \frac{3}{2} \times 2.10 \times 7.5 = 12.0$ cubic feet per second. The total discharge for the stream will be, as before, the summation of the discharges for all sections. The midsection method reduces the computations by eliminating the necessity for averaging the depths and the mean velocities in adjacent verticals.

250. Methods for Determining the Mean Velocity in a Vertical

The mean velocity in a vertical is determined by applying one of the following methods:

a. Vertical Velocity-Curve Method.

b. Two-Point Method.

c. Single-Point Method.

d. Three-Point Method.

e. Integration Method.

251. Vertical Velocity-Curve Method

The most precise, and also the most timeconsuming, method of determining the velocity in a vertical is to take a sufficient number of observations, say at every 0.1 or 0.2 of the depth, to permit plotting the vertical velocity-curve to a suitable scale. For example, at a vertical where the depth was 5.3 feet, observations were made at every 0.1 depth with the following results:

Depth in tenths	*Depth of observation (feet)*	*Velocity (feet per second)*
0.1	0.53	2.86
.2	1.06	3.01
.3	1.59	3.02
.4	2.12	2.99
.5	2.65	2.86
.6	3.18	2.63
.7	3.71	2.45
.8	4.24	2.23
.9	4.77	1.90

These values are plotted in figure 103 and the velocity curve smoothed in, the curve being extended upward to the water surface and downward to the bottom. The area enclosed between the vertical and the curve is planimetered and divided by the depth to obtain the mean velocity Vm. In this case it is 2.61 feet per second. When a planimeter is not available, the area may be closely approximated by counting full and partial squares as enclosed by the curve on the cross section paper. For comparison with other methods of determining Vm, it should be noted that the velocities at 0.2-depth and 0.8-depth are 2.62 feet per second. When the mean velocities in all verticals have been found in this manner, the discharge is computed as indicated in paragraph 251. Another alternative which is sometimes used is to plot the location of all observations on a scale drawing of the stream cross section, show the value of the velocity at each of these points, and then draw in lines of equal velocity in the same manner that contours are drawn on a topographic map from the elevations of points located by stadia. The areas between the lines of equal velocity are planimetered. The sum of the products of each such area and the average velocity for that area is the discharge of the stream. The vertical velocity-curve method is of value in determining coefficients to be applied to the results obtained by certain other methods. It is not widely used in routine measurements, not only because the time required is much greater than for other methods, but because the apparently increased precision obtainable may be

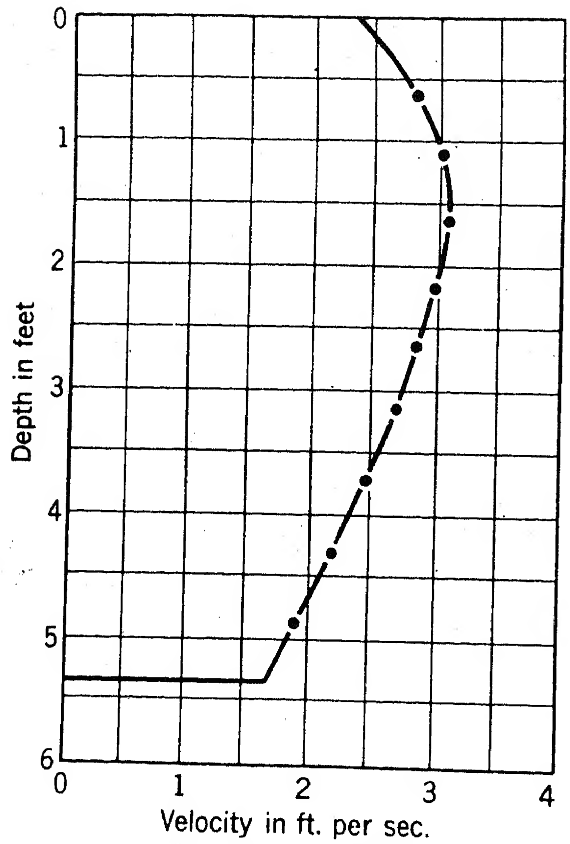


Figure 103. Vertical velocity curve.

offset by errors resulting from changes in stage during the long period of observations.

252. Two-Point Method

In the example given in paragraph 251 the average of the velocities at 0.2- and 0.8-depth was 2.62 feet per second, a value practically identical with the mean velocity obtained from the vertical velocity curve. The particular case of this example substantiates the results to be expected in any general case, for the results obtained are borne out by many studies of actual observations and by the theory that the vertical velocity curve corresponds to a section of a parabola with its axis horizontal at the point of maximum velocity. For such a condition, it may be mathematically shown that the average of the velocities at 0.2144 and 0.7886 of the depth is equal to the mean velocity. Experience indicates that the average of the velocities at 0.2- and 0.8-depth gives a closer approximation to the mean velocity than any other method except the vertical velocity-curve method when the latter is used under conditions of constant stage and steady flow. This two-point method is more widely used than any other. It should not be employed, however, with the Price meter when the depth in a vertical is less than 2 feet (0.6 m) unless a coefficient is applied. Meters of the vertical-axis cup type tend to underregister when held near the surface or near the bed of a stream. The 0.6-depth method (para. 253*a*) normally gives better results in verticals where the water is shallow.

253. Single-Point Methods

a. Six-Tenths-Depth Method. The thread of mean velocity in a vertical lies at approximately six-tenths of the depth. The example in paragraph 251 indicates a close correlation between the velocity at this depth and the mean velocity in the vertical. In general, this method does not give as good results, and is not as widely used, as the 0.2- and 0.8-depth method. It is often used in shallow depths with the Price meter but should not be used where the depth is less than 0.5 foot (0.15 m) or where the velocity is less than 0.3 foot per second (0.1 m/sec) unless a coefficient somewhat greater than unity is applied.

b. Subsurface Method. In this method the velocity is measured in the several verticals at a uniform distance below the surface, usually about 2 feet (0.6 m) and a coefficient in the order of 0.9 is applied to compute the mean velocity. The coefficient or coefficients to be applied to the computed discharge at a measuring section or to the velocity in each vertical may be obtained from vertical velocity curves as plotted for the station. Highly accurate results cannot be expected but the method is useful during flood flows when the current is so swift as to interfere with holding the meter at greater depths or when damage to the meter may result from the impact of floating debris unless it is kept in such a position that it may be quickly raised above the surface.

c. Two-Tenths-Depth Method. In this method the velocity in each vertical is measured at 0.2-depth and the discharge is computed as though this was the mean velocity in the ver-

tical. This approximate discharge divided by the cross section area gives a mean value of the 0.2-depth velocity for the section. Many experiments indicate that, for a given measuring section, the ratio between the mean 0.2-depth velocity and the true mean velocity for the section either remains constant or varies uniformly with the stage. The true discharge is found at various times by the two-point method and plotted against the approximate discharge as found by the 0.2-depth method or the true mean velocity is plotted against the 0.2-depth mean velocity at various stages. From these curves the velocity ratio or the discharge ratio for the two methods of measurement may be determined for any stage. Then at any time when the depth or velocity is such that the two-point method cannot be used, the 0.2-depth method may be employed and the computed results multiplied by the ratio corresponding to the stage of the stream.

254. Three-Point Method

In this method velocity measurements are made at 0.2, 0.6, and 0.8 of the depth below the surface. The 0.2- and 0.8-depth values may first be averaged and this result averaged with the 0.6-depth value or, if it is desired to assign greater weight to the mean of the 0.2- and 0.8-depth observations, the arithmetical mean of all three values may be used. This method is used in shallow verticals where the 0.2-depth observation is near the surface and the 0.8-depth observation is near the bottom. It is based on the assumption that the mean velocity obtained by the two-point method is too small under these conditions, that the value obtained by the 0.6-depth method is too great, and that an average of the results of the two methods more nearly represents the true value.

255. Integration Method

In the integration method the current meter is slowly lowered from the surface to the streambed and slowly raised to the surface at a uniform rate. During this process, the meter cups or vanes are acted upon by particles of water moving at velocities which vary from the surface to the streambed. Hence, if the number of revolutions are counted during the time required for a complete cycle and converted to a velocity by use of the meter-rating table, the mean velocity in the vertical should be obtained. Any vertical movement of a vertical-axis cup type meter affects the movement of the rotor to some extent so that the method is not entirely satisfactory when the Price meter is employed. If used with such a meter, the vertical movement should be at a very low rate. The method yields better results with horizontal-axis meters but is not widely used except to obtain comparisons with other methods.

256. Meter Measurements in Streams That Are Swift and Deep

In rivers that have high velocity and great depth, the current meter and weight will drift downstream from a vertical position. Thus the sounding as made with the cable and weight is greater than the actual depth of the water and, if it is desired to place the meter at 0.8-depth (for example), the meter will actually be at less than this depth when the computed length of cable has been payed out. For accurate measurements the vertical angle made by the cable should be measured and corrections applied to obtain true depths in the vertical and proper placement of the meter. For such conditions, reference is made to the data contained in U.S. Geological Survey Water-Supply Paper 888, Stream-Gaging Procedure.

257. Measurements in Ice-Covered Streams

When the measuring section is covered by ice, measurements of velocity may be made by cutting holes at each vertical with an ice chisel or axe and lowering the current meter through the holes (fig. 104). The distance from the water surface in the holes to the bottom of the ice must be measured with an L-shaped graduated rod and the effective depth determined by subtracting this distance from the total depth. The distance is usually measured at both the upstream and downstream sides of the hole and a mean value taken. If there is a layer of frazil ice below the surface ice, its thickness must also be taken into account in determining the effective depth. The lower limit of such an ice layer is found by raising the meter upward from the bottom of the stream until a level is reached where it fails to regis-

ter any velocity. Pulsations of the water level in the holes are likely to be observed. A sufficient number of depth readings should be taken to establish the mean effective depth. When the effective depth is greater than 2 feet (0.6 m), the two-point method may be used for determining the velocity in the vertical. For lesser depths, the readings may be taken at 0.5 depth and a coefficient of 0.88 applied, or taken at 0.6 depth and a coefficient of 0.92 used. These coefficients should, if possible, be determined for the individual measuring section by defining one or two vertical velocity curves in the deeper verticals. In subfreezing temperatures the meter should be kept under water as much as possible. Between measurements in successive

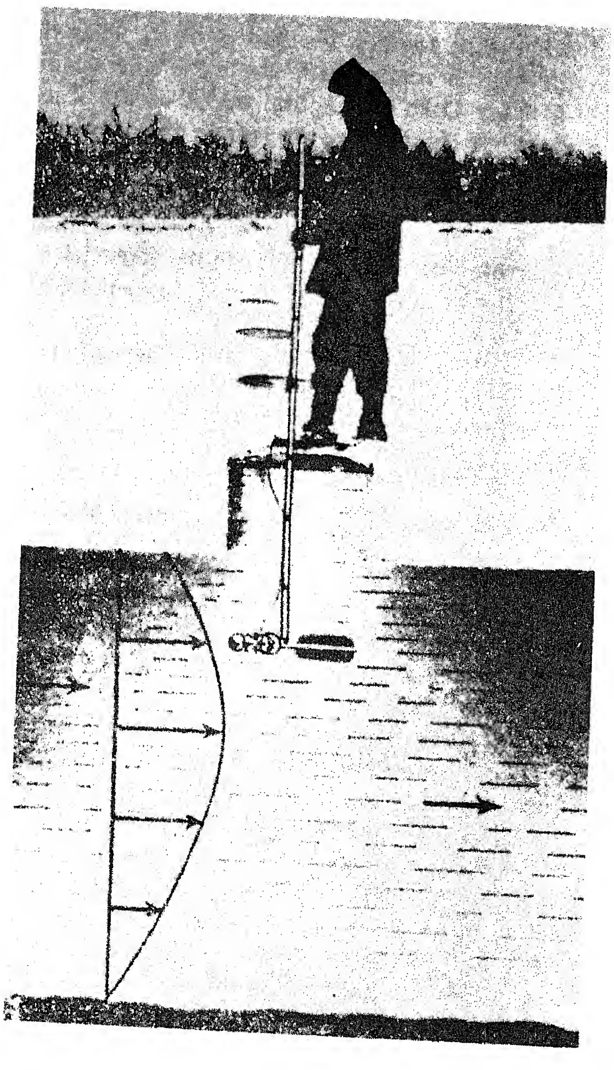


Figure 104. Streamflow measurement through ice cover.

verticals it may be necessary to warm the meter near a fire to prevent formation of ice in the meter bearings, and to eliminate frazil ice from the rotor cups. Since ice conditions at a control may vary considerably from day to day, discharge records are considered to be approximate during the period of ice effect.

258. Discharge-Measurement Notes

a. Notes for Open Water Measurement. Figure 105 shows typical discharge measurements using the two-point method except in shallow water near one bank where the 0.6-depth method was used. The discharge has been computed by the mean-section method (para. 251*a*). The first column shows the horizontal distance across the stream to each vertical, measured from an initial point on one bank. The difference in successive values in this column gives the width of each section (col 12). For the use of column 2 see *b* below. Column 3 shows the depth in each vertical. Column 4 shows the depth of observation, 0.2, 0.8, or 0.6 of the depth. Column 5 shows the number of revolutions counted in the time indicated by the figures given in column 6. From these figures, the velocity at each point (col 7) is taken from the meter-rating table (table VII). The mean velocity in the vertical (col 8) is the velocity at 0.6 depth or the average of the velocities at 0.2 and 0.8 depth. When, as in this case, the discharge is computed by the mean-section method, the mean velocity in the section (col 9) is the average of the mean velocities in adjacent verticals. The area of the section (col 10) is the mean depth (the average of the depths at adjacent verticals) shown in column 11 multiplied by the width of the section (col 12). The discharge for the section (col 13) is the product of the mean velocity in the section and its area. The total discharge is the sum of the discharges of all sections across the stream. Columns 9 through 12 are so arranged that terms which are to be multiplied together are in adjacent columns. Thus, the product of the terms in columns 11 and 12 gives the area which is entered in column 10. This, in turn, is multiplied by the mean velocity in column 9 to give the discharge entered in column 13.

b. Notes for Angle Coefficient. The angle coefficient (col 2, fig. 105) is used when the di-

Discharge Measurement Notes ~ Clearwater River at Dover ~ 6 May 19												
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Dist from Initial point feet	Angle Coefficient	Depth feet	Depth of Observation feet	Revolutions	Time Secs.	Velocity At point — Feet per Sec.	Mean in Vertical	Mean in Section	Area Sq. Ft.	Mean Depth Feet	Width Feet	Discharge c.f.s.
0		0	0	—	—	0	0					
								0.91	7.0	1.40	5	6.4
5		2.8	0.6	40	44	2.03	1.81					
			2.2	35	49	1.60		2.23	14.0	2.80	5	31.2
10		2.8	0.6	55	45	2.72	2.65					
			2.2	48	50	2.58		2.73	13.5	2.70	5	36.8
15		2.6	0.5	60	42	3.18	2.80					
			2.1	50	46	2.43		2.85	13.2	2.65	5	37.6
20		2.7	0.5	60	41	3.26	2.90					
			2.2	50	44	2.53						
50		3.6	0.7	70	42	3.72	3.02					
			2.9	50	48	2.33		2.36	16.5	3.30	5	38.9
55		3.0	0.6	40	48	1.87	1.70					
			2.4	30	44	1.53		1.32	11.5	2.30	5	5.2
60		1.6	0.3	20	46	0.99	0.94					
			1.3	20	52	0.88		0.47	2.1	1.05	2	1.0
62		0.5	0.3	—	—	0	0					
		Discharge computed by mean-section method.									Total	493.2

Figure 105. Discharge-measurement notes (open water).

rection of flow at a vertical is not perpendicular to the measuring section and it is necessary to obtain the component of velocity which is normal to the measuring section. This condition usually occurs where measurements are made from a bridge which crosses the stream on a skew. In this case, a cable-supported meter will swing parallel to the direction of the current and the horizontal axis of the meter will make an angle less than 90° with the measuring section. The natural cosine of this angle, known as the *angle coefficient*, must be determined so that the measured velocity can be multiplied by this coefficient to determine the component of velocity normal to the measuring section. Printed forms for stream-gaging notes used by agencies doing much of this work are provided with a diagram for the graphical determination of the angle coefficient. When the notes are being kept in the military-issue fieldbook, this diagram is not available and the angle coefficient is best determined by the following method as indicated in figure 106.

(1) Hold the 1-foot mark of a tape or rule graduated in hundredths of a foot or the 1-m mark of a tape graduated in cm, at a convenient point *A* on the bridge.

(2) Swing the zero end of the tape to *B* so that *AB* is parallel to the direction of the current as indicated by the meter.

(3) Draw a short line *BC* parallel to the direction of the measuring section.

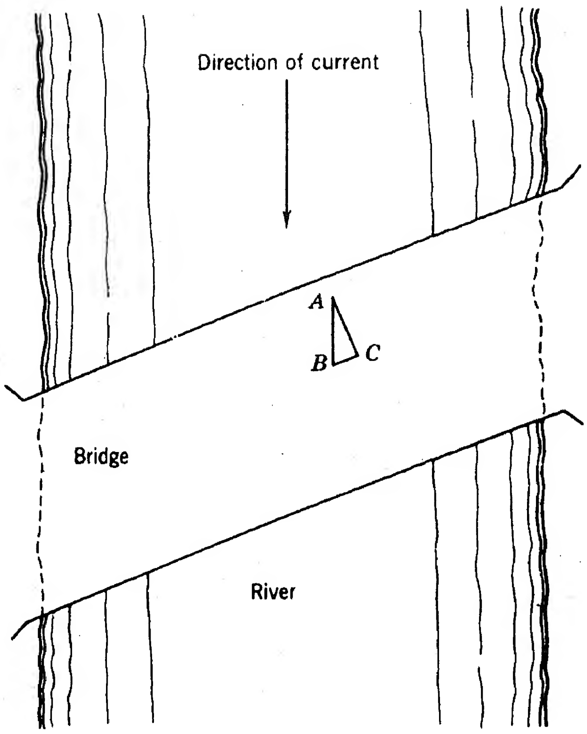


Figure 106. Determination of angle coefficient.

(4) Holding the 1-footmark fixed at *A*, swing the tape until it is perpendicular to *BC*. The distance *AC* in hundredths of a foot, will represent the angle coefficient.

c. Notes for Measurements Under Ice Cover. Figure 107 shows typical measurements for an ice-covered stream. It shows the second page of notes for a stream of such width that three pages were required to include measurements at all verticals. Column 1 is the same as for the open water notes (fig. 105). Column 2 shows two values: the first, at any vertical, is the thickness of the ice; the second is the distance from the water surface to the bottom of the ice. At 80 feet from the initial point (fig. 107), the ice thickness was 1.5 feet. The distance from the water surface to the bottom of the ice was 1.4 feet. Column 3 also shows two values—the first is the total depth of water to the bed of the stream; the second is the effective depth. This equals the total depth of water less the distance from the water surface to the bottom of the ice. The effective depth is circled to indicate that this is the depth to use in computing the area. The remaining columns are identical with those used for open water notes. In this case the discharges through the several sections have been computed by the midsection method (para. 258*b*).

d. Special Notes.

(1) The notes for open water or ice-cover measurements should be preceded by a page in the notebook devoted to general and special data. These include—

(*a*) Name of stream and location of station.

(*b*) Date.

(*c*) Party.

(*d*) Width and area of stream, mean velocity, mean gage height, and total discharge.

(*e*) Method used, number of sections, and method of suspension of meter (cable or rod).

(*f*) Gage readings of inside and outside gages at start and end of measurements and, if variations are noted, at periodic intervals during the measurements.

(*g*) Comparison of measurement with station-rating curve or table.

(*h*) Type of measurement, whether taken from upstream or downstream side of bridge, by wading, through ice, from cableway, or from boat.

(*i*) Weather and other pertinent remarks.

(*j*) Spin of meter before and after measurement.

(2) Most of these entries are self-explanatory. The last one, spin, requires further mention. The meter is placed with its shaft in a vertical position (in a location protected from air currents) and the bucket wheel is given a quick turn to start it spinning. The time until it stops spinning is read on a stop watch and the manner in which it comes to a stop, whether abrupt or gradual, is also noted. A new meter will spin for about 4 minutes before the rotor comes to rest. The instrument is in satisfactory condition if the wheel spins for 1 minute or more and

Discharge Measurement Notes ~ Ice Cover – Wide River at Great Falls 6 Feb. 19__												
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Dist. from Initial Point feet	Thickness of ice W.S. to bot. ice	Total depth of water Effective depth	Depth of meter below water surface	Rev-olu-tions	Time in secs.	Velocity – At point	Feet per Second Mean in Vertical	Mean in Section	Area Sq. Ft.	Mean Depth Feet	Width Feet	Discharges c.f.s.
								Brought forward				160.6
80	1.5	4.6	0.2	5	54	0.22	0.17	0.17	32	3.2	10	5.4
	1.4	3.2	0.8	2	40	0.12						
90	1.6	6.9	0.2	10	45	0.50	0.40	0.40	54	5.4	10	21.6
	1.5	5.4	0.8	7	51	0.31						
100	1.5	7.0	0.2	15	57	0.58	0.54	0.54	56	5.6	10	30.2
	1.4	5.6	0.8	10	44	0.51						
110	1.5	7.0	0.2	10	39	0.57	0.52	0.52	56	5.6	10	29.1
	1.4	5.6	0.8	10	47	0.48						
120	1.5	6.8	0.2	10	39	0.57	0.52	0.52	53	5.3	10	27.7
	1.5	5.3	0.8	10	46	0.48						
130	1.6	6.4	0.2	15	47	0.70	0.61	0.61	48	4.8	10	29.3
	1.6	4.8	0.8	10	43	0.52						
140	1.5	6.4	0.2	15	42	0.78	0.64	0.64	49	4.9	10	31.4
	1.5	4.9	0.8	10	44	0.51						
150	1.4	6.0	0.2	20	45	0.97	0.82	0.82	46	4.6	10	37.7
	1.4	4.6	0.8	15	50	0.66						
										Total		373.0
		Discharge computed by mid-section method.										

Figure 107. Discharge measurement notes (ice cover).

comes to a gradual stop. A meter which stops abruptly or spins for less than 1 minute requires reconditioning and recalibration. A marked change in the spin-time before and after measurements indicates possible damage to the bearings and throws doubt on the accuracy of the measurements.

259. Sources of Error in Discharge Measurements

Once a station-rating curve has been determined from a series of current-meter measurements at various stages, the discharge at any time is taken from the curve by noting the discharge corresponding to the stage as read on the water-stage recorder. Check-measurements and inspections of the station are required periodically to eliminate or minimize errors from such sources as changes in cross section resulting from scouring or deposition and erroneous stage readings on the recorder resulting from clogged intake pipes. Errors may also result from backwater effects so extensive as to drown out the control section, from ice effects at the control, and from a marked rising or falling in the stage of the stream. The station-rating curve normally is constructed as a smooth curve. As such, it represents conditions of uniform flow at a given stage of the stream. When the stream is rising, the water slope will be steeper than when the flow is uniform and, for a given stage, the discharge will be greater than it will be under

uniform-flow conditions at that same stage. The converse is true when the stream is falling. In certain instances separate rating curves are established for rising and falling stages.

260. Velocity-Area Measurements Using Floats

Figure 108 shows a typical layout for float measurements. Two transits are set up at convenient points *A* and *B*, several hundred feet apart. The distance *AB* is measured, usually by stadia. Right angles are turned from the base *AB* with both transits, and flags or range poles are set at *C* and *D*. A boatman places the float in the water at a point such as *E*. The transitman at *A* with vernier reading 0°, sights flag *C* and clamps his instrument. The transitman at *B* sets 0° on flag *D* and then loosens the upper motion, turning the telescope until the float is in the field of view. As the float moves slowly downstream, this instrumentman keeps it on the vertical hair by slowly turning the telescope. When the float nears line *AC* the timekeeper calls a warning. As it passes the vertical crosshair of transit *A*, this instrumentman calls "Time" and the timekeeper starts a stopwatch while the transitman at *B* clamps his upper motion on the line *BF* and reads the angle *DBF*. A similar process is repeated as the float approaches *G*, the roles of the two instrumentmen being reversed. A graphical solution for the distance *FG* will suffice and the angles *DBF* and *CAG* need be read no closer than they can be plotted by protractor. The diagram is plotted to a large scale so that *FG* may be scaled as closely as *AB* was measured by stadia. The distance *FG* divided by the elapsed time as measured with the stopwatch will give the velocity. The boatman will recover the float and measurements will be taken at other points across the width of the stream. The product of the mean velocity and cross section area gives the discharge. Current-meter measurements of velocity and discharge are far superior to float measurements. The latter are useful for reconnaissance survey purposes or for measurements during floods when high water, high velocities, and floating drift limit the use of the current meter.

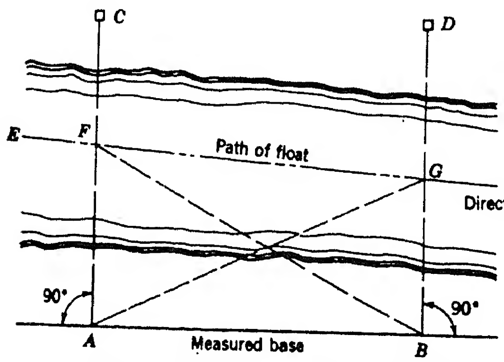


Figure 108. Float measurements.

261. Weir Measurements

A weir in its simplest form consists of a notch in the top of the vertical side of a reservoir through which water is flowing. Equations, based upon laboratory tests, have been developed for weirs of various shapes and types which express the velocity in terms of the *head* on the weir. This head *H* (fig. 109) is the height of the water surface above the weir crest, measured at a point upstream from the weir just above the point where the water level starts to drop as it approaches the crest. This point where the head is measured is usually taken at a distance of about 3 *H* upstream from the crest. Weirs may be rectangular, triangular, trapezoidal, or parabolic in form. Weirs are frequently used to measure discharge in small streams where the velocities and depths are not suitable for current-meter use, and in irrigation ditches, power canals, large sewers, and other manmade hydraulic channels in which weirs can be easily installed.

262. Measurements Required at Weir Installations

a. Weir formulas give the velocity as a func-

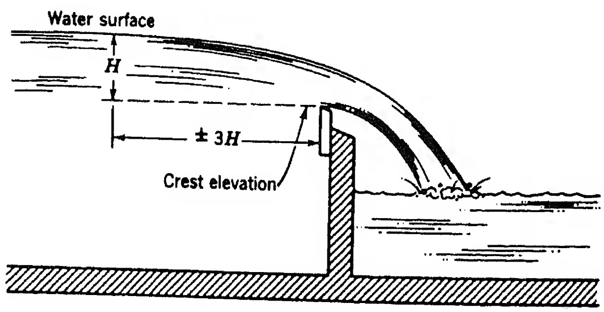


Figure 109. Sharp-crested rectangular weir.

tion of the head on the crest. The area of the overflowing stream can be expressed in terms of the length of the weir crest b and the head H or, in the case of a triangular weir of known notch angle, in terms of H alone. The area and velocity terms are usually combined in weir formulas to give the discharge directly. The measurements required at weir installations to permit discharge computations include—

(1) The static head H on the weir (fig. 109).

(2) The crest length b of rectangular or trapezoidal weirs (figs. 110, 115).

(3) The angle or angles of side slopes for triangular or trapezoidal weirs (figs. 114, 115).

(4) The elevation of the crest above the bottom of the approach channel (fig. 111).

(5) The area of the approach channel.

(6) The velocity of flow in the approach channel if this is appreciable (para. 264).

(7) The number of end contractions, if any (para. 263).

b. It is also necessary to determine if the water level downstream is above or below the crest. If it is above the crest, the downstream head H' (fig. 112) must be measured. It must be known whether the weir is sharp-crested or broad-crested and, in the latter case, the shape of the crest must be determined.

263. Rectangular Weirs

a. Sharp-Crested Weir. A sharp-crested rectangular weir has a vertical upstream face with a beveled edge (fig. 109). With such a weir the under edge of the overflowing sheet of water or nappe touches only the upstream edge of the crest. There is free circulation of air beneath the nappe and there is a definite vertical contraction in the thickness of the nappe as the water leaves the crest.

b. Contracted Weir. When the weir notch extends for only a portion of the width of the approach channel (fig. 110), the water filaments approaching the edges of the weir from the sides of the channel must turn to pass through the notch. As a result, there will be contractions at the edges, that is, the cross-

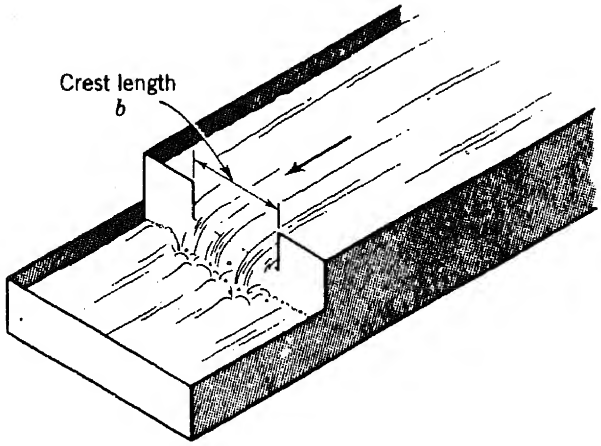


Figure 110. Contracted weir.

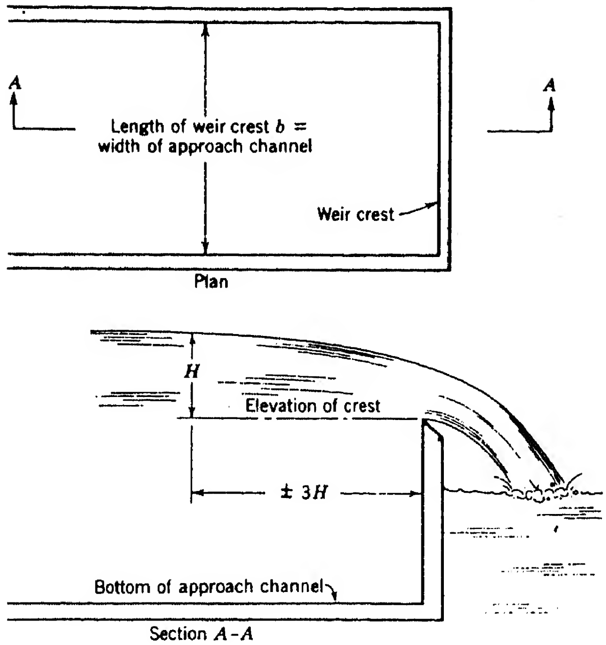


Figure 111. Suppressed weir.

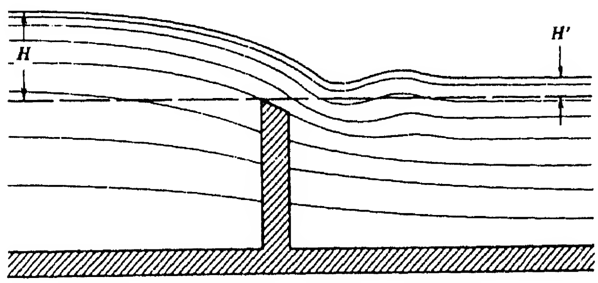


Figure 112. Submerged weir.

channel length of the nappe will be somewhat less than the length of the weir crest. Such a weir is known as a contracted weir. There may be two end contractions, as shown in figure 110, or only one where one edge of the weir notch is flush with a side of the channel.

c. Suppressed Weir. When the weir notch extends for the full width of the approach channel (fig. 111), these contractions are suppressed (are not formed) and the weir is known as a *suppressed weir.*

d. Submerged Weir. When the water level downstream from the weir stands at a head H' (fig. 112) above the elevation of the crest the weir is known as a *submerged weir.* As the water level downstream rises above the crest the vertical contraction of the nappe observed in the weir with free overflow (fig. 109) is suppressed and, for small values of H', the discharge is greater than it is for a weir with free overflow. As H' is increased still further, the effect of backwater on the downstream side causes the discharge to diminish. Thus, for this weir, the discharge is a function not only of H but of H' as well. Submerged weirs are chiefly used where it is necessary to minimize losses in head.

e. Broad-Crested Weirs. Weirs are usually constructed with sharp crests since the greater part of the laboratory work to determine coefficients for the various weir formulas has been performed on weirs of that type. In the field various hydraulic structures such as the spillways of dams or overflow spillways in levees may be used for discharge measurements. The weir crests in these cases are broad and of various shapes. Flat crests decrease the discharge up to the point where the head becomes so great that the overflowing water jumps clear of the downstream edge when the discharge becomes the same as for a sharp-crested weir. Under low heads the discharge of a broad flat-crested weir may be as much as 25 percent less than that over a sharp-crested weir under the same head. A rounded upstream edge of the crest of a flat-topped weir increases the discharge. If the entire crest is rounded, the discharge is reduced for low heads but may increase with high heads as the curve of the crest becomes more nearly the same as the normal curve of the underside of the nappe. While a limited amount of laboratory work has been performed on broad-crested weirs of various shapes to determine weir coefficients for various heads (para. 264), these coefficients apply only to weirs which conform in dimensions to those used in the experiments and only within a very limited range of heads. Where such structures are used in the field it is generally more satisfactory to measure the actual discharges under various heads with a current meter and to deduce the coefficients which apply, in a basic weir formula, to the particular structure under the range of heads actually encountered.

264. Discharge Formulas for Rectangular Weirs

Many different weir formulas have been developed for the computation of discharge over rectangular weirs. A number of those which are in common use are covered in *a* through *e* below.

a. Hamilton Smith Formula. The Hamilton Smith formula

$$Q = cbH^{3/2}$$

is basically simple in form and easily applied. In this equation:

- Q = the discharge in second-feet.
- c = a weir coefficient based on experimental data and varying with the head on the weir and the length of the weir.
- b = length of weir crest in feet.
- H = head on weir crest in feet.

Values of c for suppressed weirs for a given head and length of weir may be interpolated from table VIII. Similar values for contracted weirs having two end contractions are given in table IX. The formula gives good results within the rather limited range of heads and lengths of weir covered in the tables. When the velocity in the approach channel is appreciable, H should be increased to a value Hv which will correct the measured static head H for the effect of the velocity of approach. Hv is then used in the formula instead of H. For a suppressed weir, $Hv = H + 1.33\ h$. For a contracted weir, $Hv = H + 1.4\ h$. In the term, $h = \frac{V^2}{2_g} = \frac{\frac{Q^2}{a}}{2_g}$ where V is the mean velocity in the approach channel, a is the area of the

approach channel in square feet measured at he gage where H is determined, and g is the acceleration of gravity. This value of H represents the kinetic head of the approaching water resulting from its velocity. In determining Hv, Q is first computed approximately by using H in the formula, neglecting the velocity of approach and finding an approximate value for h. The approximate value of Hv obtained in this way may then be used to determine a new value for Q and a still closer value for h from which final values of Hv and Q may be computed.

b. Francis Formula. Francis developed the formula

$$Q = 3.33\ bH^{3/2}$$

for suppressed weirs without consideration of velocity of approach. Where the latter term is appreciable, the formula becomes

$$Q = 3.33\ b\ ((H + h)^{3/2} - h^{3/2})$$

For contracted weirs, Francis substituted the term b — 0.1 nH) for b in the above formulas, n in this case being the number of end contractions.

c. Bazin Formula. The Bazin formula, applicable to suppressed weirs for heads ranging up to 6 feet, is

$$Q = 8.02\ mbH^{3/2}$$

in which m =

$$\left(0.405 + \frac{0.00984}{H}\right)\left(1 + 0.55\right)\left(\frac{H}{G + H}\right)^2$$

and G is the height of the weir crest above the bottom of the approach channel. Table X gives values of Q per foot of width of weir for various values of H and G.

d. Multipliers for Broad-Crested Weirs. The above formulas apply to sharp-crested weirs with free overflow. The flow over a broad-crested weir may be computed by multiplying the discharge over a sharp-crested weir of the same height and length by a factor which varies with the shape or width of the crest. Figure 113 shows weirs of various shapes and dimensions which have been the subject of full-sized model experiment. Tables XI, XII, and XIII contain the multipliers developed for these weirs or spillways under various heads. Where spillways are encountered in the field which approximate one of these types in shape and dimensions, these multipliers may be used when it is necessary to obtain an estimate of the discharge quickly. When time permits it is desirable to obtain a multiplier for the actual structure from discharges measured with a current meter.

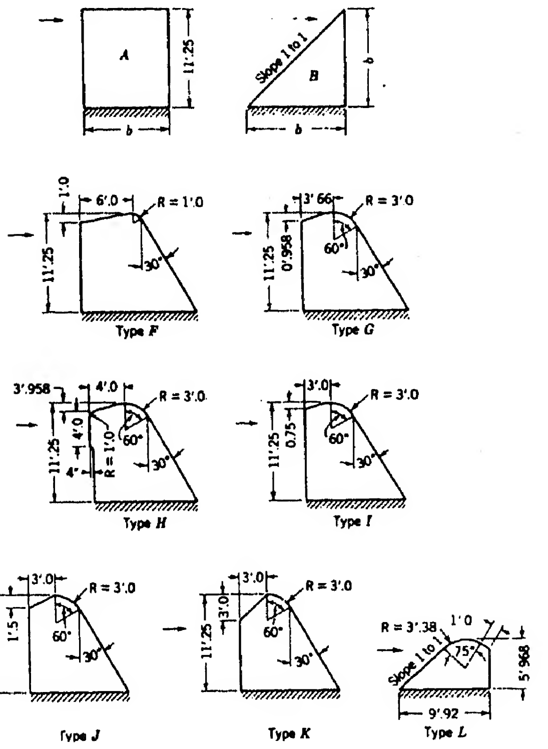


Figure 113. Forms of weirs and dams.

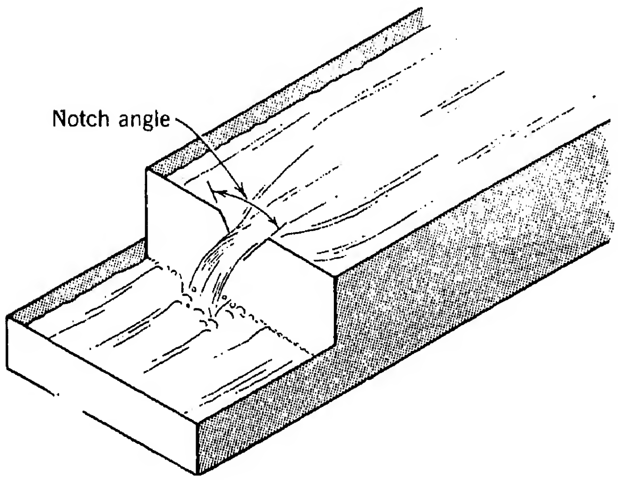


Figure 114. Triangular weirs.

Table VIII. Hamilton Smith's Coefficients for Suppressed Weirs

Hamilton Smith's coefficients for weirs with contraction suppressed at both ends, for use in the formula $Q=cbH^{3/2}$.

H=head in feet	b=length of weir, in feet								
	19	1	10	7	5	4	*3	*2	*0.66
0.1	3.515	3.515	3.520	3.520	3.526	------	------	------	3.611
0.15	3.440	3.445	3.445	3.451	3.451	3.461	3.472	3.488	3.542
0.2	3.397	3.403	3.408	3.408	3.413	3.429	3.435	3.450	3.510
0.25	3.371	3.376	3.381	3.386	3.392	3.403	3.413	3.429	3.494
0.3	3.349	3.354	3.360	3.365	3.376	3.386	3.403	3.418	3.483
0.4	3.322	3.328	3.333	3.344	3.360	3.371	3.386	3.403	3.478
0.5	3.312	3.317	3.322	3.338	3.354	3.371	3.386	3.408	3.478
0.6	3.306	3.312	3.317	3.333	3.354	3.371	3.392	3.413	3.483
0.7	3.306	3.312	3.317	3.338	3.360	3.376	3.397	3.424	3.494
0.8	3.306	3.317	3.322	3.344	3.365	3.386	3.408	3.441	3.510
0.9	3.312	3.317	3.328	3.354	3.375	3.397	3.418	3.451	------
1.0	3.312	3.322	3.338	3.360	3.386	3.408	3.429	3.467	------
1.1	3.317	3.328	3.344	3.371	3.397	3.419	3.445	------	------
1.2	3.317	3.333	3.349	3.381	3.403	3.429	3.456	------	------
1.3	3.322	3.338	3.360	3.386	3.413	3.440	3.467	------	------
1.4	3.328	3.344	3.365	3.392	3.424	3.445	------	------	------
1.5	3.328	3.344	3.371	3.403	3.429	3.456	------	------	------
1.6	3.333	3.349	3.376	3.408	3.435	3.461	------	------	------
1.7	3.333	3.349	3.381	3.413	------	------	------	------	------
2.0	------	------	------	------	------	------	------	------	------

* Approximate.

Table IX. Hamilton Smith's Coefficients for Contracted Weirs

Hamilton Smith's coefficients for weirs with two complete end contractions, for use in the formula $Q=cbH^{3/2}$.

H=head in feet	b=length of weir, in feet										
	0.66	*1	2	2.6	3	4	5	7	10	15	19
0.1	3.381	3.419	3.456	3.478	3.488	3.494	3.494	3.499	3.504	3.504	3.510
0.15	3.312	3.344	3.392	3.408	3.413	3.419	3.424	3.424	3.429	3.435	3.435
0.2	3.269	3.306	3.349	3.365	3.371	3.376	3.376	3.381	3.386	3.392	3.392
0.25	3.237	3.274	3.322	3.333	3.338	3.344	3.349	3.354	3.360	3.360	3.365
0.3	3.215	3.253	3.296	3.306	3.312	3.322	3.322	3.333	3.338	3.338	3.344
0.4	3.183	3.215	3.258	3.274	3.280	3.285	3.290	3.301	3.306	3.312	3.317
0.5	3.156	3.189	3.237	3.247	3.253	3.264	3.269	3.280	3.290	3.295	3.301
0.6	3.140	3.172	3.215	3.231	3.237	3.247	3.253	3.269	3.280	3.285	3.290
0.7	3.130	3.156	3.199	3.210	3.226	3.231	3.242	3.258	3.274	3.280	3.285
0.8	------	------	3.183	3.199	3.215	3.221	3.231	3.247	3.269	3.274	3.280
0.9	------	------	3.167	3.189	3.199	3.210	3.226	3.242	3.258	3.269	3.274
1.0	------	------	3.156	3.172	3.183	3.199	3.215	3.231	3.253	3.264	3.269
1.1	------	------	3.140	3.162	3.172	3.189	3.205	3.226	3.242	3.258	3.264
1.2	------	------	3.130	3.151	3.162	3.178	3.194	3.215	3.237	3.253	3.264
1.3	------	------	3.114	3.135	3.151	3.167	3.199	3.205	3.231	3.247	3.258
1.4	------	------	3.103	3.124	3.140	3.156	3.178	3.199	3.221	3.242	3.258
1.5	------	------	------	3.114	3.130	3.151	3.167	3.189	3.215	3.237	3.253
1.6	------	------	------	3.103	3.114	3.140	3.162	3.183	3.210	3.231	3.247
1.7	------	------	------	------	------	------	------	3.178	3.205	3.226	3.247
2.0	------	------	------	------	------	------	------	------	------	------	------

* Approximate.

Table X. Discharge in Cubic Feet Per Second Per Foot of Length Over Sharp-Edged Vertical Weirs Without End Contractions

[Computed by Bazin's formula]

Head (H) feet	Height in feet of crest of weir above bottom of channel of approach													
	G=2	G=3	G=4	G=5	G=6	G=7	G=8	G=9	G=10	G=12	G=16	G=20	G=25	G=30
0.2	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
0.3	.58	.58	.58	.58	.58	.58	.58	.58	.58	.58	.58	.58	.58	.58
0.4	.88	.88	.88	.87	.87	.87	.87	.87	.87	.87	.87	.87	.87	.87
0.5	1.23	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.20	1.20	1.20
0.6	1.62	1.59	1.59	1.58	1.58	1.58	1.58	1.57	1.57	1.57	1.57	1.57	1.57	1.57
0.7	2.04	2.01	1.99	1.98	1.98	1.98	1.98	1.97	1.97	1.97	1.97	1.97	1.97	1.97
0.8	2.50	2.45	2.43	2.42	2.41	2.41	2.41	2.40	2.40	2.40	2.40	2.40	2.40	2.40
0.9	3.00	2.93	2.90	2.88	2.88	2.87	2.86	2.86	2.86	2.86	2.86	2.85	2.85	2.85
1.0	3.53	3.44	3.40	3.38	3.36	3.36	3.35	3.35	3.34	3.34	3.33	3.33	3.33	3.33
1.2	4.68	4.55	4.48	4.47	4.42	4.41	4.40	4.39	4.38	4.38	4.37	4.36	4.36	4.36
1.4	5.99	5.78	5.68	5.62	5.58	5.56	5.54	5.53	5.52	5.51	5.49	5.49	5.48	5.48
1.5	6.68	6.44	6.30	6.23	6.20	6.18	6.16	6.14	6.13	6.12	6.11	6.10	6.09	6.09
1.6	7.40	7.12	6.97	6.89	6.84	6.80	6.78	6.76	6.74	6.73	6.71	6.69	6.69	6.69
1.8	8.93	8.56	8.37	8.25	8.18	8.13	8.09	8.07	8.05	8.02	7.99	7.98	7.97	7.96
2.0	10.58	10.12	9.87	9.72	9.62	9.55	9.51	9.47	9.44	9.40	9.36	9.34	9.33	9.32
2.2	12.34	11.77	11.46	11.27	11.14	11.06	10.99	10.95	10.91	10.86	10.81	10.78	10.76	10.75
2.4	14.20	13.53	13.15	12.91	12.75	12.64	12.56	12.50	12.45	12.39	12.32	12.28	12.25	12.24
2.5	15.17	14.45	14.03	13.76	13.59	13.47	13.38	13.31	13.26	13.18	13.10	13.06	13.03	13.01
2.6	16.16	15.38	14.92	14.63	14.44	14.30	14.20	14.13	14.07	13.99	13.90	13.85	13.82	13.80
2.8	18.23	17.23	16.79	16.44	16.21	16.04	15.92	15.83	15.76	15.66	15.54	15.48	15.44	15.42
3.0	20.39	19.36	18.74	18.33	18.06	17.86	17.71	17.60	17.52	17.39	17.25	17.18	17.13	17.10
3.2	22.64	21.48	20.77	20.31	19.98	19.75	19.58	19.45	19.34	19.19	19.02	18.93	18.87	18.83
3.4	24.98	23.70	22.89	22.36	21.99	21.72	21.52	21.36	21.24	21.06	20.86	20.75	20.68	20.63
3.5	26.20	24.83	24.00	23.43	23.01	22.73	22.48	22.38	22.22	22.00	21.83	21.69	21.62	21.60
3.6	27.41	25.99	25.09	24.49	24.06	23.75	23.52	23.34	23.20	22.99	22.75	22.62	22.53	22.48
3.8	29.94	28.38	27.38	26.70	26.22	25.87	25.60	25.39	25.23	24.99	24.71	24.56	24.45	24.39
4.0	32.54	30.84	29.74	28.99	28.45	28.05	27.74	27.51	27.32	27.05	26.72	26.55	26.42	26.35
4.2	35.22	33.39	32.18	31.35	30.75	30.30	29.96	29.69	29.48	29.17	28.79	28.59	28.45	28.36
4.4	37.99	36.01	34.70	33.78	33.12	32.62	32.24	31.94	31.70	31.34	30.92	30.66	30.52	30.42
4.6	40.83	38.71	37.29	36.29	35.56	35.01	34.58	34.25	33.98	33.58	33.10	32.84	32.65	32.53
4.8	43.75	41.49	39.96	38.87	38.07	37.45	37.00	36.62	36.33	35.88	35.35	35.05	34.83	34.70
5.0	46.71	44.31	42.67	41.49	40.62	39.96	39.44	39.03	38.70	38.21	37.61	37.28	37.03	36.88
5.2	49.81	47.27	45.50	44.23	43.29	42.57	42.01	41.56	41.20	40.65	39.99	39.61	39.33	39.17
5.4	52.94	50.23	48.38	47.02	46.00	45.22	44.60	44.11	43.71	43.12	42.38	41.96	41.66	41.47
5.6	56.15	53.33	51.34	49.88	48.79	47.94	47.28	46.74	46.31	45.65	44.82	44.38	44.04	43.83
5.8	59.42	56.45	54.34	52.79	51.62	50.71	49.99	49.41	48.94	48.22	47.33	46.83	46.45	46.22
6.0	62.77	59.65	57.43	55.78	54.53	53.55	52.78	52.15	51.64	50.86	49.90	49.34	48.92	48.67

e. Formula for Submerged Weirs. When a weir is submerged (fig. 112) the basic formula becomes

$$Q = 3.33\ b\ (n'\ H)^{3/2}$$

where n' is a number depending upon the ratio of H', the downstream head, to H. Values of n' are given in table XIV.

265. Triangular Weirs

A triangular weir (fig. 114) is frequently used on streams where the discharge may be, at times, very small. The coefficient varies with the inclination of the sides of the notch. For a sharp-crested weir with 1:1 side slopes (notch angle 90°) the basic formula is $Q = 2.6\ H^{5/2}$.

266. Trapezoidal Weir

The trapezoidal weir (fig. 115) is a combination of a rectangular weir with the two halves of a triangular weir. The discharge through the two triangular ends tends to compensate for the loss in discharge resulting from the end contractions of a rectangular contracted weir. If this balance were perfect the trapezoidal weir would have the same discharge as a rectangular suppressed weir of the same

Table XI. Multipliers for Flat-Topped Weirs (A, fig. 113)

Head (H) feet	Width of flat crest in feet							
	b=0.48	b=0.93	b=1.65	b=3.17	b=5.89	b=8.98	b=12.24	b=16.30
0.5	0.902	0.830	0.819	0.797	0.785	0.783	0.783	0.783
1.0	.972	.904	.879	.812	.800	.798	.795	.792
1.5	1.000	.957	.910	.821	.807	.803	.802	.797
2.0	1.000	.989	.925	.821	.805	.800	.798	.795
2.5	1.000	1.000	.932	.816	.800	.795	.792	.789
3.0	1.000	1.000	.938	.813	.796	.791	.787	.784
3.5	1.000	1.000	.942	.810	.793	.787	.783	.780
4.0	1.000	1.000	.947	.808	.790	.783	.780	.777

Table XII. Multipliers of Weirs of Triangular Cross Section (B, fig. 113)

Head (H) feet	b=6.65	b=11.25	Head (H) feet	b=6.65	b=11.25
0.5	1.060	1.060	2.5	1.076	1.096
1.0	1.079	1.079	3.0	1.067	1.095
1.5	1.091	1.092	3.5	1.060	1.094
2.0	1.086	1.097	4.0	1.054	1.093

Table XIII. Multipliers for Compound Weirs (fig. 113)

Head (H) feet	Type F	Type G	Type H	Type I	Type J	Type K	Type L
0.5	0.964	0.932	0.934	0.968	0.971	0.971	0.971
1.0	1.026	.982	1.000	1.008	1.040	1.040	.983
1.5	1.064	1.015	1.040	1.032	1.083	1.092	1.022
2.0	1.066	1.031	1.061	1.041	1.105	1.126	1.040
2.5	1.025	1.038	1.073	1.043	1.118	1.146	1.057
3.0	.992	1.044	1.082	1.044	1.128	1.163	1.072
3.5	.966	1.049	1.090	1.045	1.136	1.177	1.085
4.0	.944	1.053	1.097	1.046	1.144	1.190	1.097

Table XIV. Values of n' for Various Values of $\frac{H'}{H}$ (fig. 112)

H'/H	n'	H'/H	n'	H'/H	n'	H'/H	n'
0.00	1.000	0.18	0.989	0.38	0.935	0.58	0.856
.01	1.004	.20	.985	.40	.929	.60	.846
.02	1.006	.22	.980	.42	.922	.62	.836
.04	1.007	.24	.975	.44	.915	.64	.824
.06	1.007	.26	.970	.46	.908	.66	.813
.08	1.006	.28	.964	.48	.900	.70	.787
.10	1.005	.30	.959	.50	.892	.75	.750
.12	1.002	.32	.953	.52	.884	.80	.703
.14	.998	.34	.947	.54	.875	.90	.574
.16	.994	.36	.941	.56	.866	1.00	.000

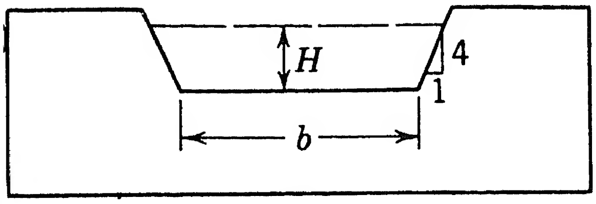


Figure 115. Cipolletti trapezoidal weir.

length of crest. This balance is approximately realized when the sides of the notch have a slope of one horizontal to four vertical. A weir with these side slopes is known as a *Cipolletti* weir. The usual formula for such a weir, without velocity of approach, is $Q = 3.367\ bH^{3/2}$.

267. Construction of Weirs

To construct a weir—

a. Select a site with high banks so that all overflow must pass over the weir crest at all stages of the stream.

b. Select a site where bank and channel material are such that leakage around and under the weir can be prevented.

c. Locate weir perpendicular to axis of stream.

d. Make upstream face vertical; the crest sharp-edged and truly horizontal; and the height of the crest above the bed of the channel at least twice the maximum expected head to minimize the effect of velocity of approach.

e. Be sure there is sufficient fall below the crest to permit air circulation on the under side of the weir unless a submerged weir is to be used.

f. Construct weirs across small streams of wood sheet piling, providing a metal crest having a bevel on the downstream edge; construct larger and more permanent weirs of concrete with a sheet metal crest; use sheet metal for weirs in small artificial channels.

g. Build all weirs solidly and brace them well to provide sufficient rigidity to prevent vibration.

h. Seal all joints in the structure to prevent leakage.

i. Place an apron of planks, rocks, or concrete on the downstream side to prevent scour.

268. Use of Dams as Weirs

The spillway of a dam may be used as a weir provided it has a level crest free of obstructions, sufficient height to provide free fall at all stages of backwater below the dam, little or no leakage through, around, or under the structure, a crest and cross section of such shape that a satisfactory coefficient for use in the formula $Q = cbH^{3/2}$ is either known or can be determined, and provided means are at hand for measuring any flow which does not pass over the structure but is taken off through turbine intakes or through pipelines. U.S. Geological Survey Water-Supply Paper 200, Weir Experiments, Coefficients and Formulas, contains values of coefficients which may be used for such structures pending actual determination of the coefficients for various heads by current-meter measurements.

269. Discharge by Slope-Area Measurements

The slope-area method involves the selection of a suitable longitudinal reach of the stream in which the measurements are made, the determination of the mean area, perimeter, slope, and roughness of the channel within the reach, and the substitution of these data in the Chezy formula

$$Q = CA\sqrt{RS}$$

where Q is the discharge in second-feet.

A is the mean cross section area in the reach.

R is the mean hydraulic radius. For any cross section, the hydraulic radius equals the ratio of the area to the wetted perimeter (that portion of the cross section perimeter consisting of banks and channel bed which is wet by the flowing water).

S is the slope of the energy gradient (para. 273).

C is a coefficient depending upon the roughness of the channel and the hydraulic radius (para. 270).

270. Determination of C in Chezy Formula

The Manning formula $C = \dfrac{1.486\ R^{1/6}}{n}$ is commonly used to determine the coefficient C.

Table XV. Values of n for Canals and Ditches and for Natural Streams [1]

Surface	Condition of stream bed			
	Best	Good	Fair	Bad
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	[2] 0.0225	0.025
Rock cuts, smooth and uniform	.025	.030	[2] .033	.035
Rock cuts, jagged and irregular	.035	.040	.045	------
Winding sluggish canals	.0225	[2] .025	.0275	.030
Dredged earth channels	.025	[2] .0275	.030	.033
Canals with rough, stony beds, weeds on earth banks	.025	.030	[2] .035	.040
Earth bottom, rubble sides	.028	[2] .030	.033	.035
Natural stream channels:				
1. Clean, straight bank, full stage, no rifts or deep pools	.025	.0275	.030	.033
2. Same as (1) but some weeds and stones	.030	.033	.035	.040
3. Winding, some pools and shoals, clean	.033	.035	.040	.045
4. Same as (3), lower stages, more ineffective slopes and sections	.040	.045	.050	.055
5. Same as (3), some weeds and stones	.035	.040	.045	.050
6. Same as (4), stony sections	.045	.050	.055	.060
7. Sluggish river reaches, rather weedy or with very deep pools	.050	.060	.070	.080
8. Very weedy reaches	.075	.100	.125	.150

[1] As used by Horton. [2] Values commonly used in design.

In this formula, n is a roughness factor to be taken from table XV and R is the hydraulic radius. Other formulas for C and tables of values of C for various values of n and R will be found in the issue text on hydraulics (King, Hydraulics Handbook).

271. Use and Limitations of Slope-Area Method

The slope-area method is often used in determining flow in open or closed conduits where the channel is uniform and the roughness of the channel lining is relatively constant. It is used in natural channels where, because of backwater and other effects, there are such variations in the surface slope that the stage discharge relationship is unstable. The limitations of the method lie in the difficulty of measuring flat slopes with precision and in selecting a proper value of the roughness coefficient n. Bed conditions which might appear good to one observer might be classed as fair by another and there may be a difference in judgment as to what constitutes a "rather weedy reach" and what constitutes a "very weedy reach" (table XV). In natural channels the results obtained are only approximate except where they are supplemented by current-meter determinations of Q from which reliable values of C for the given reach may be obtained for various surface slopes.

272. Selection of Reach for Slope-Area Measurements

The reach selected for slope-area measurements should fulfill the following criteria:

a. Straight, with a length of 1,000 feet or more.

b. Little variation either in cross-section area of channel or in the mean velocity of flow from point to point along the reach.

c. A uniform drop in the water surface between the upper and lower ends of the reach. If possible, this drop should amount to 1 foot or more.

273. Measurements at Slope-Area Stations

a. Area. Where a suitable reach has been located, measurements of cross section area are made at a sufficient number of points to obtain a representative value for the mean area. One such measurement may suffice in an artificial channel of uniform width and depth whereas a natural channel may show sufficient variation to require a considerable number of measurements at controlling points.

b. Hydraulic Radius. The hydraulic radius is determined at each cross section where the area is measured. In artificial channels this ratio of area to wetted perimeter can be computed from the measurements of width and depth. In natural streams it is convenient to plot each cross

section to scale and to scale the wetted perimeter as a series of short straight segments from the diagram. A mean value of R is computed for the reach.

c. Slope.

(1) *Definition and discussion.* The slope S in the Chezy formula is the slope of the *energy gradient.* It is frequently designated Se to differentiate it from the slope of the water surface Ss. The elevation of the energy gradient of a stream above a given datum is the sum of its elevation head and its velocity head. The elevation head represents the potential energy of the stream, in foot-pounds per pound of water, resulting from its elevation above the datum. Similarly, the velocity head, $v^2/2g$, represents the kinetic energy, in the same units, resulting from the velocity of flow. The sum of the two gives the total energy per pound of water. This is known as the *specific energy.* Figure 116 shows a sloping stream channel in which the flow is uniform. The channel area, depth of flow, and velocity remain constant as the flow passes from A to C. Referred to the datum BC, the specific energy of the stream at A equals AB plus d plus $v^2/2g$. At C, the specific energy is d plus $v^2/2g$. Thus, for this case, the change in specific energy or drop in the energy gradient (SeL) equals the drop in the channel bed AB in the length of reach L. Since the depth remains constant, the drop in the energy gradient equals the drop in the water surface and Se equals Ss. Figure 117 represents a channel in which the flow is nonuniform. The depth at A is larger than at C while the velocity at C is greater than that at A. The figure indicates that the slope of the energy gradient differs, in this instance, from that of the water surface. The water surface slope Ss is measured in the field ((2) below). The slope of the energy gradient is then found from the formula:

$$Se = Ss - (v_2^2 - v_1^2)/2gL$$

where Se is the slope of the energy gradient, Ss is the measured slope of the water surface, v_1 is the mean velocity at section A at the upper end of the reach, v_2 is the mean velocity at section C at the lower end of the reach, L is the length of the reach, and g is the acceleration due to gravity. If v_1 is larger than v_2 the formula becomes:

$$Se = Ss + (v_1^2 - v_2^2)/2gL$$

When the downstream velocity is less than that upstream and there is a transformation of kinetic energy to potential energy, it normally is assumed that the actual recovery is about 50 percent of the theoretical recovery, that is, only half of the second term in the formula immediately above is added to Ss to obtain Se. In either case, the value of Se is used in place of S in the Chezy formula. Direct measurements of v_1 and v_2 are not made. An approximate value of Q is determined from the Chezy formula using the surface slope. This is divided by the areas at A and C (fig. 117) to obtain approximate values of v_1 and v_2. The term Se is determined from these, a closer value of Q is computed and new values of v_1 and v_2 are determined from which a final value of Q is computed.

(2) *Measurement of water-surface slope.* Determine the slope of the water surface in the reach in the following manner.

(*a*) Install gages, preferably on both banks, at both the upper and lower ends of the reach.

(*b*) Determine the elevation of the zero of each gage by differential leveling. Great care is necessary in this operation since the slopes normally are flat and a small error in leveling will have a large effect upon the value obtained for the discharge.

(*c*) Make simultaneous readings of the gage heights of all gages.

(*d*) Determine the mean water level for

each end of the reach.

(*e*) Divide the difference in water levels at the two ends of the reach by the length of the reach to obtain the slope of the water surface *Ss*.

(*f*) Where the flow appears to be substantially uniform, use this surface slope in the Chezy formula. Where there is an apparent difference in the velocities at the two ends of the reach, compute *Se* as indicated in (1) above.

d. Roughness Factor. Examine carefully in the field the condition of the channel throughout the length of the reach. On the basis of this examination, select an appropriate value for the roughness factor *n* from table XV.

274. Orifice Measurements of Discharge

An orifice is an opening, usually circular, square, or rectangular in shape, in the side of a dam or other structure, through which water is discharged under the action of the head of water above the center of the opening. The orifice may discharge freely into the air or it may discharge below the tail water level downstream. In the latter case it is known as a submerged orifice. The discharge through an orifice is computed from the formula:

$$Q = c_d a \sqrt{2gh}$$

where Q is the discharge in second-feet.

a is the area of the orifice in square feet.

h is the effective head in feet.

g is the acceleration of gravity in feet per second.[2]

c_d is the coefficient of discharge of the orifice. For free discharge the effective head is the height of the water surface on the upstream side above the center of the orifice. For submerged orifices the effective head is the difference between the water levels on the upstream and downstream sides. Experiments indicate tnat c_d is greater for low heads than for high ones, greater for rectangular orifices than for square ones, and greater for the latter than for circular ones. The coefficient for a submerged orifice is usually about 1 percent less

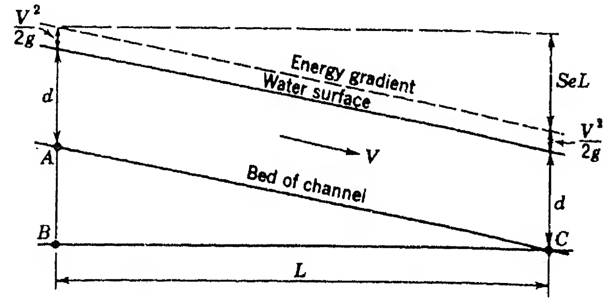


Figure 116. Energy gradient for channel with uniform flow.

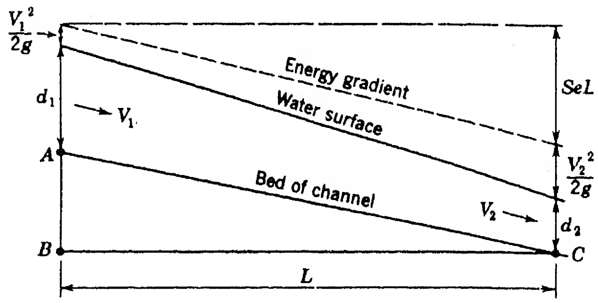


Figure 117. Energy gradient for channel with nonuniform flow.

than for a free-flowing orifice under the same head. The range in coefficients is not large. A value of 0.61 may be used for rough computations. Complete tables of coefficients are found in the *Hydraulics Handbook*, by King. The orifice is used for discharging measured quantities of water through dams, locks, tide gates, canals, and other artificial waterways.

275. Special Methods of Measuring Discharge

There are a number of special methods and devices for measuring discharge through conduits under pressure and in open channels. These include the Venturi meter, various types of registering water meters, the Parshall flume, nozzles, volumetric methods, the salt-velocity and salt dilution methods, and the use of colored dyes. These methods involve special devices not usually available to hydrographic survey parties or require special construction. For a discussion of these methods, reference is made to the *Hydraulics Handbook*, by King.

276. Dredging Operations

Dredging must be carried out in widening or deepening channels in harbors, rivers, or canals; in providing adequate depth of water alongside piers and in anchorages; and in the construction of foundations for bridge piers and other waterfront structures. Dipper and clamshell dredges remove the material to scows which are towed to a deep water dumping ground. Hopper dredges remove the material to hoppers within the hull of the dredge for transportation to sea. Hydraulic dredges pump the material to low-lying areas ashore where it is used as fill. Computations of the yardage involved are required in estimating the size of a project, the time necessary to complete it, and interim progress on the work. Both in-place measurements and scow measurements are used as a basis for computations. Measurements made in place, before dredging, will generally show a volume about 10 percent less than that obtained by scow measurements.

277. In-Place Measurements

The measurement of material in place involves the taking of soundings at regularly spaced intervals over the area of the work. The spacing of the soundings will depend upon the smoothness or irregularity of the bottom and upon the difficulty of removal of the material. Soundings are made before and after dredging and the volume of material removed is computed by either the borrow-pit method or by the end-area method of cross sections (TM 5-233). The soundings taken before dredging are also used for the preliminary estimate of the magnitude of the work contemplated. Soundings taken in tidal waters or in rivers at different stages of the stream must be reduced to a common datum. Borings must be taken to define the character of the material to be removed to permit the selection of proper dredging equipment and to provide data for estimates. In hydraulic dredging it is sometimes more convenient to measure the material in place after it has been pumped as fill onto diked areas ashore. In this case provision must be made for determining and allowing for any settlement of the original land surface under the weight of the fill. Where cross sections are used in the end-area method of computing the volume the areas of the cross sections may be computed or the cross sections may be plotted to scale and the areas determined by planimeter or by counting the unit squares enclosed by the outline of the cross section.

278. Scow Measurements

a. By Volume. Materials removed by dipper dredge are usually loaded into scows provided with a number of pockets or hoppers having bottom-opening doors for convenience in dumping. The pockets are measured and, for each scow used on the project, a table of data is prepared giving the volume of each pocket when filled to various levels below the deck coaming. When a scow has been loaded, the inspector counts the full pockets and, in the case of pockets which are partially full, measures down from the coaming to the point where he estimates the material would come if leveled off. The total volume loaded is then determined from the table. These measurements should be made just before towing to the dumping ground. If a scow is loaded one day and moored overnight before being dumped, some of the material may leak out through the bottom doors which may not fit tightly. If such leakage occurs over the working area, the material will be redredged. It should not be counted twice. Dredged sand and gravel which is to be used for concrete aggregate is loaded onto deck scows and is smoothly mounded with straight side slopes and flat top. The ends of such scows are usually provided with vertical partitions to hold the material so there are no slopes at the ends. The volume is readily determined by taking measurements to obtain the area of the trapezoidal cross section and multiplying this by the length of the scow between end partition.

b. By Displacement. The volume of rock removed is generally determined by in-place measurements but, in exposed locations subject to high seas, this method may not be feasible. When the broken rock is placed loosely in hopper pockets or on deck scows, its solid volume is obtained by determining its weight and

dividing by the weight per cubic yard. The weight of the material is determined by measuring the displacement of the scow before and after loading. Since any floating body, such as a scow, will displace a volume of water of the same weight as the body, the change in displacement provides a measure of the load. Figure 118 shows a loaded deck scow 100 feet long on deck, 76 feet long on the bottom, with 12-foot vertical sides, and with a width of 20 feet. Assume that the draft in sea water, when light, is 4 feet and that the draft when loaded is 10 feet. The waterline length for the two conditions of loading are determined, as indicated in figure 119, to be 84 feet and 96 feet respectively. The volume of water displaced when light will then be—

$$\left(\frac{84 + 76}{2}\right) 4 \times 20 = 6{,}400 \text{ cubic feet.}$$

The displacement loaded will be—

$$\left(\frac{96 + 76}{2}\right) = 17{,}200 \text{ cubic feet.}$$

The difference is 10,800 cubic feet. Since sea water weighs 64 pounds per cubic foot, the weight of the water displaced by the load is $10{,}800 \times 64 = 691{,}000$ pounds. If the rock is gneiss which weighs 5,400 pounds per cubic yard, the volume of the rock (as a solid) is

$$\frac{691{,}000}{5{,}400} = 128 \text{ cubic yards.}$$

In computing barge loads of rock in hopper scows by the displacement method, care must be taken to allow for water below the waterline in the hoppers. This requires determination of the proportionate volume of solid material in the hopper load to the volume of the hopper, usually 55 to 63 percent. Corrections must also be made for additional bilge water within the scow at the time the loaded draft is measured.

279. Capacity of Reservoirs and Lakes

a. Contour Method. The capacity of a reservoir is always computed prior to construction and filling but, particularly where the entering stream is silt-laden, periodic resurveys are necessary to determine the remaining volume of water storage. Soundings are taken at suitable intervals along range lines as indicated in figure 120. Ranges and soundings are lo-

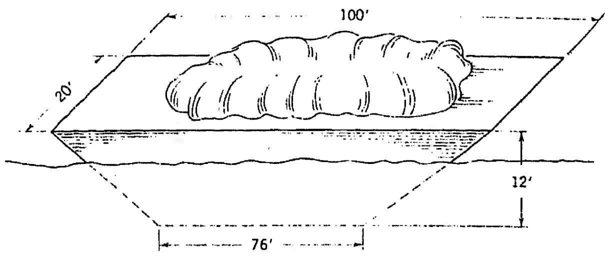


Figure 118. Loaded deck scow.

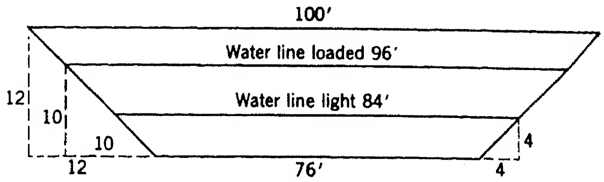


Figure 119. Waterline lengths of scow.

cated by procedures which are described in paragraphs 246 through 305. The spacing of the ranges and of the soundings along them must be sufficiently close to clearly define the slopes of the reservoir bed. In general, a spacing of 200 to 400 feet will be satisfactory unless the soundings indicate a marked variation in depths between adjacent measurements. The elevation of the water surface is determined by leveling from a bench mark to a gage, and the elevation of the bottom at each sounding is computed and plotted on a map of the reservoir. These elevations are used to sketch in subaqueous contours as indicated by the dotted lines in figure 120. The area enclosed by each contour is then planimetered. If elevation 270 represents the minimum level to which the water can be drawn down by the outlet works, the volume below this elevation is disregarded. The volume in cubic feet between the 270- and 280-foot contours is the average of the areas enclosed by these contours, measured in square feet, multiplied by the contour interval of 10 feet. The volumes between successive contours up to the maximum water level are found in a similar manner and are successfully added to the preceding accumulated total to give a table of volumes of storage for various reservoir levels. These values are plotted to produce a curve showing the elevation-storage relationship, from which the stor-

age for any elevation of the water surface can be determined. Volumes are usually expressed in cubic feet, acre feet (43,560 cubic feet), or billions of gallons.

b. Cross Section Method. Instead of plotting the subaqueous contours, cross sections of the reservoir may be plotted for each range such as *A–A*, *B–B*, and so forth (fig. 120). The areas of all cross sections are then determined by planimeter and the volume of water between successive ranges determined by averaging the areas of the two cross sections and multiplying by the distance between ranges. The summation of all partial volumes gives the total volume. The areas on the cross sections are determined for different elevations of the water surface by drawing horizontal lines on the plotted cross sections at, for example, elevation 270, 280, 290, and 300 and planimetering the respective areas. Thus an elevation-storage curve can be plotted as in the preceding method. The capacity of lakes is determined in the same manner. A lake, however, occupies a natural depression and is controlled in elevation by a rock reef or other geologic formation at the outlet, whereas a reservoir is usually formed by a dam across a stream valley which slopes downward toward the dam. Thus the storage in a lake which is useful for water supply or water power may represent a very small part of the total volume. In a reservoir it may be possible to utilize substantially the entire capacity.

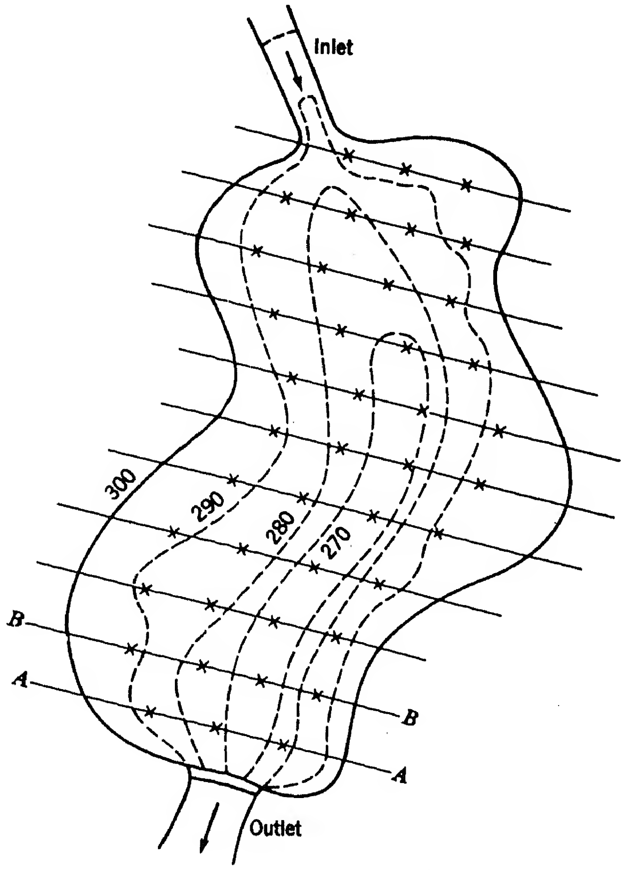


Figure 120. Soundings for determining reservoir volume.

280. Water Resources Surveys

There is a military requirement for surveys of the surface water and ground water resources of various regions. Such surveys are conducted by hydrologic teams. They include the gathering of data relating to precipitation, water losses, the variation in runoff of surface streams, measurements of ground water levels, and estimates of the quantity of water stored as snow or ice on a catchment area during the winter months. The mission of survey personnel is one of support to the hydrologic teams. The survey party prepares such base maps as may be required and gathers such data concerning precipitation, ground water levels, snow storage, and stream flow as may be directed by the hydrologist.

Section VII. SUBAQUEOUS SURVEYS

281. Introduction

Subaqueous surveys are carried out to gather data required for port construction, development, improvement, or rehabilitation; to supply information necessary for other types of subaqueous construction; to measure the capacities of lakes and reservoirs; to obtain data needed for the preparation of charts; to furnish information on the configuration of the beds of bodies of water and on the character of the materials underlying them; to measure the depths and the area of waterways at stream-gaging stations; and to determine the quantities

and depths in dredging operations. Such surveys extend from one-man operations in measuring the cross section of a shallow stream to field parties surveying harbors and coastal waters. Surveys carried out at a considerable distance offshore require the use of ships provided with much specialized equipment and many instruments, and manned by large crews.

282. General Reconnaissance

A subaqueous survey of any considerable magnitude should be preceded by a general reconnaissance to determine the scope of the work to be accomplished and the methods by which it may best be done. This reconnaissance should include studies of possible schemes of triangulation control of the area (TM 5–441) and of feasible methods of locating the shoreline and prominent landmarks. It should develop general information on the roads and other means of transportation and communication within the area, campsites and possible billeting facilities, the location of sheltered anchorages for sounding craft, and similar data which will aid in the planning or execution of the detailed surveys.

283. General Survey Procedures

An extensive subaqueous survey normally involves the following survey procedures:

a. Horizontal control by triangulation and traverse is extended over the survey area.

b. Field observations are adjusted and the geodetic positions or military-grid coordinates of all control stations are computed.

c. These positions are plotted on projection sheets which have been laid out to appropriate scales and which cover the survey area.

d. Leveling operations for vertical control are carried out.

e. Tide stations are established.

f. Topographic surveys are run along the shore.

g. The data from all these surveys are plotted on field sheets which are issued to the hydrographic parties.

h. Soundings are plotted on these sheets from data obtained by the hydrographic parties. When the size of the sounding vessel permits, the data are plotted while the work progresses so that the extent of the coverage is apparent at all times and necessary additional soundings can be taken before the vessel leaves the area.

284. Special Types of Subaqueous Surveys

Under the urgencies of various military situations, all of these normal survey procedures may not be feasible. Special types of surveys which may be required include:

a. Surveys in Advance of Control. Many subaqueous surveys of small lakes, harbors, or short stretches of coast are made to satisfy local and immediate needs and are carried out in advance of, or simultaneously with, the control surveys. Ranges to guide the boats are set on shore, and transits are located at convenient points for sighting the positions of the sounding boats. Prominent objects such as flagpoles, chimney stacks, lighthouses, and buoys are selected as signals for sextant locations from the boats. Thus, many soundings can be taken and located while the control surveys are being executed and while topographic parties are locating the shoreline, the topographic and physical features, and the transit stations and ranges used in the hydrographic work. For certain purposes geodetic control will be unnecessary and the entire survey can be tied to a few local control stations. Even in surveys of rather extensive areas the work can be adjusted for for the effect of the earth's curvature when all local stations have been tied into the main triangulation network. In many military operations, a fully restituted photomap can be prepared in advance and used as a base map for the survey.

b. Reconnaissance Survey. A reconnaissance survey is a rapidly executed preliminary survey of a region to provide advance information to meet immediate military needs. It normally is made to a comparatively small scale, is usually uncontrolled by triangulation, and may be little more than a sketch of the area. Relatively few soundings are taken and they provide no more than a general idea of the water depths and the location of the main channels. The legend on the resulting chart should clearly indicate the approximate nature of the information shown.

c. Running Survey. A running survey is an exploratory survey of an unknown or hostile

coast made from shipboard to determine the general form of the coast and the nature of the area. The result of such surveys are far from accurate and a scale of 1: 100,000 or smaller is sufficient to show the limited details. The ship proceeds along the coast at a safe distance from shore, its position being fixed by dead reckoning and periodic astronomic observations. As the vessel proceeds on course, soundings are taken at intervals and angles or bearings are measured to all identifiable points on the shore and to conspicuous objects inland, the times of the several observations being recorded to correlate the data with the dead reckoning of the ship's position. All off-lying islands and rocks should be located and sketches made showing their general shape and the general shape of the topography ashore. When two or more bearings have been taken to an identifiable point from successive positions of the ship, the point is plotted by graphical triangulation. A series of such points will serve to delineate the general outline of the shore. It is particularly important to thus locate as many points as possible within bays and harbors to correctly portray the shape of the coast within these indentations. When conditions permit, a more accurate survey is made by anchoring buoys during the run of the ship along definite courses, the distances and azimuths between these buoys being determined by taut wires stretching between them, and by sun azimuths taken from the ship. As soon as it is practicable to put parties ashore the positions of such buoys as are visible from the shore may also be determined by intersection from two located points on shore. When the positions of the buoys have been determined and plotted, detailed hydrographic surveys may be made inshore, using small boats and fixing their successive positions by sextant angles using the buoys as control points.

d. Survey of an Isolated Harbor. In military operations it is frequently necessary to complete a survey of a harbor or anchorage before making subaqueous surveys throughout the region and before geodetic control has been established by triangulation extending over the general area. A local control network is established with a measured base and horizontal angles determined with transits or sextants. For rapid work, planetable triangulation may be used. Positions and azimuths are based on astronomic observations. Vertical control for topography may be based upon an assumed datum or upon a short series of tide observations. A few of the triangulation stations and bench marks are permanently marked and referenced for future connection with the main control for the general area. The hydrographic survey is then tied to this local control. When time is limited, the main channels and the anchorage area should be thoroughly sounded, with soundings in other areas taken at sufficient intervals to give a general idea of the depths.

285. Accuracy of Measurements

The accuracy of measurements in subaqueous survey operations is dependent upon the purpose of the survey, the scale used in plotting the data, the time and personnel available, the prevailing conditions at the time the work is carried out, and the military conditions and requirements.

a. Control. Precision measurements are used for base lines, angle measurements, astronomic work, and leveling for the general control system extending along a long coastline. On the other hand, a base measured by stadia or taut-wire, planetable, or sextant triangulation, and ordinary differential leveling will often suffice for control of local surveys.

b. Scales. The scales control the accuracy of plotting horizontal distances and angles. No greater precision is warranted in making the measurements used for plotting individual soundings that can be shown in plotting at the scale used. Distances to 1 foot and angles to the nearest 5 minutes normally will suffice and rougher measurements will be suitable for the smaller scales. Maps or charts for special purposes may be executed on larger or smaller scales than those in (1) through (4) below, depending upon the amount of detail to be shown (FM 30-5). Scales in current military use for charts employed in amphibious operations include—

(1) Naval approach charts, 1: 72,000.

(2) Naval bombardment charts, 1: 36,000.

(3) Naval air-support charts, 1: 50,000.

(4) Amphibious beach-assault special large scale maps, 1: 10,000.

c. Depth Measurements. Depth measurements are customarily given in feet or in fathoms. Published charts of Atlantic coastal waters normally show depths in feet. Fathoms and feet are in general use on published charts of the Pacific coast. To avoid mistakes, the two units should not be used on different sections of the same chart. Depths are recorded to the nearest one-half foot at controlling points on navigable bars, in channels less than 42 feet in depth, over shoals, rocks, and other dangers which lie at depths less than 42 feet, in shoal waters and inside passages, and adjacent to the low water line.

d. Position of Sounding. The accuracy of measurement of angles and distances used in locating soundings is somewhat dependent upon the method used. In general, such measurements need be taken no closer than they can be plotted on the scale used for the chart. In subaqueous construction, such as a cofferdam for a bridge pier, both horizontal distance and depth measurements may be required to $\frac{1}{10}$ foot (3 cm) to enable divers to properly position and wedge the bottom tiers of bracing required to hold the cofferdam walls in position subsequent to dewatering.

286. Guides for Geographic Nomenclature

a. Names of Control Stations. For identification purposes and for convenient reference, all control stations used in connection with subaqueous surveys are commonly assigned names. Descriptive, geographic, or personal names having a definite connection with the locality are used in preference to names selected arbitrarily. For example; a station on Cooper Ridge might be called Cooper; a station on the Howard farm might be called Howard; a station on an unnamed headland might be called Headland. Where a station is so located that it cannot be designated readily as indicated above, an arbitrarily selected name may be applied to it. There are certain definite advantages which accrue from having such an arbitrary name reflect, in a general way, the relative importance of the station named. Names of five or more letters are usually assigned to triangulation or traverse stations. Those of four letters are used to designate marked topographic stations, shoran stations, electronic position indicator stations, and temporary topographic and hydrographic stations used by ships. Buoys and temporary topographic and hydrographic stations used in small-boat surveys are generally designated by three-letter names. It is of the utmost importance that the same name should not be used for two stations in the same locality and particularly within the limits of the same survey. To avoid possible duplication, the names should be recorded on a list as they are assigned.

b. Names of Geographic Features. Correct geographic names are as essential on a nautical chart as on a topographic map. Each place, all prominent works of man, and all important hydrographic and topographic features should be properly identified by the name which they are known in the locality, correctly spelled. It is awkward and militarily unsound to have to refer to prominent geographic features by the use of long descriptive phrases. The use of latitude and longitude or of map coordinates is sound military practice but normally is used to confirm the location of named places. No other feature of a map or chart is more useful to the user than authentic place names. The hydrographer or topographer must ascertain the names which are in general local usage to define the important features in an area. These names may apply to channels, sloughs, rivers, inlets, reefs, rocks, banks, shoals, lakes, islands, hills, mountains, towns and hamlets, railways, highways, and other important works of man. Names should not be copied from old maps or charts without verification. Descriptive reports covering a hydrographic survey should contain evidence confirming the names applied to prominent features.

c. Jurisdiction of U.S. Board on Geographical Names. The U.S. Board on Geographical Names was established in 1890 to assure uniform usage as to geographic nomenclature on maps and charts produced by Federal agencies and to decide unsettled questions on geographic names. Two or more names for the same feature may be in local or established usage. Such cases are

submitted, accompanied by a statement of all information obtained by the investigator, to the board for decision. Additional duties of the board involve decisions on disputed spellings and application of geographic names; the determination, change, and fixing of place names in the United States and its possessions; and review of newly assigned names suggested for use on Federal maps and charts in publication.

287. Project Specifications

Project specifications define the limits of a project area and indicate the operations which are to be performed, the methods to be used, and the degree of accuracy to be attained. They will cover these matters with respect to control, topographic, and subaqueous surveys, and any tidal observations or measurements of tidal currents which are included in the project. With reference to the subaqueous survey, the specifications will indicate the limits of the work both along shore and offshore, required overlap for adjacent surveys, the scales to be used in different parts of the area, the maximum spacing between lines of soundings, the percentage of cross lines to be run as a check on the accuracy of the work and the adequacy of coverage, the maximum intervals between soundings, the frequency with which samples of the bottom material are to be taken, and such other requirements as are deemed necessary.

288. Finishing the Subaqueous Survey

A hydrographic survey is not complete until there is assurance that all features pertinent to the survey have been located and the least depths over shoals determined.

a. Locating Reefs and Shoals. The existence of shoals and the characteristics of subaqueous topography may be disclosed by depth measurements and other indications obtained during the systematic survey; by use of the wire drag or sweep; from reports of local seafarers; from visual evidence in clear, calm water; by the presence of eddies in tidal currents; from the presence of kelp; from the presence of schools of shoal water fish and of the sea birds which feed on them; and from evidence disclosed by aerial photographs (fig. 121). Once a shoal has been located, it should be marked by one or more buoys and a detailed survey made of the

Figure 121. Shoals disclosed by aerial photograph.

area. The least depth over an extensive shoal is usually obtained from closely spaced soundings taken along radiating ranges from a centrally located buoy.

b. Locating Buoys and Rocks Awash. All buoys and prominent surface objects within the area must be located by strong sextant fixes or by angles from shore stations so that they may be correctly positioned on the finished chart. Every isolated bare rock or rock awash within the survey area must be located and its height determined. If it is possible to land on the rock a strong sextant fix can be obtained. Otherwise, the rock may be cut-in from shore stations or from successive positions of the sounding boat.

c. Sea Bottom Characteristics. The character of the sea bottom is determined at frequent and regular intervals as the sounding work proceeds. Usually samples are taken at every fixed sounding and sometimes more often in harbors, channels, and anchorages. These data are valuable in connection with port construction, developments, improvements, and rehabilitation; in locating satisfactory anchorages; and in dredging operations and subaqueous construction. Samples of certain bottom materials may be brought to the surface by arming the

lead with tallow or soap. For other bottom materials, a specially devised snapper with two-spoon-shaped jaws closed by a spring is attached to the lead. On striking the bottom the weight of the lead forces the spring to close the jaws. Neither the armed lead nor the snapper will bring up specimens from a rocky bottom. This is best indicated by the feel of the leadline as the lead strikes, or by abrasions on the lead. The consistency of other bottom materials may also be determined by the feel of the leadline. Both the armed lead and the snapper take samples from the surface layer of the bottom and give no indication of the material below. For subaqueous construction it is frequently necessary to obtain core samples to a considerable depth.

d. Plotting Topographic and Hydrographic Detail. The final map or chart of the survey area will show topographic data taken from the planetable sheets which were completed by the topographic party and data from the boat sheets and sounding records compiled by the hydrographic party. Normally, the data from a number of different boat sheets and sounding records are first plotted on what is known as the *smooth sheet*, which is a record of the hydrographic work in the project area. Both the hydrographic and topographic sheets are usually to a larger scale than that used for the finished map. The field sheets can be inked and photographically reduced to the final scale for convenience in transferring the data to the final worksheet from which the chart is to be produced. This final drawing is laid out on a large sheet of high-grade drawing paper mounted on muslin. Projection lines are drawn and all control stations are carefully plotted.

(1) *Topographic details.* Methods of plotting topographic details are given in TM 5–232. The details shown will comprise the shore line; contours representing the topography some distance back from the shore; all important physical features and works of man; wooded areas; marsh lands; single trees and other objects which serve as landmarks when viewed from seaward; and all channel range markers, lights, and other navigational aids located on the shore.

(2) *Hydrographic details.* The hydrographic details comprise the high and low water lines; all navigational aids located offshore; the location of isolated rocks, shoals, rocks awash, wrecks, and other dangers; the limits and depths of channels; the location of anchorages; areas of cable crossings; selected soundings which show representative depths and all controlling points where major changes in the slope of the bottom occur, as well as the least depths over shoals; abbreviations indicating the character of the bottom; depth contours or tinted areas to show the changing depth of the water; and a compass rose showing the variation of the magnetic needle and its annual rate of change. It is unnecessary to show on the final chart all of the soundings which appear on the boat sheets or in the sounding records. The depth may be substantially constant over an extensive area and a few well chosen soundings will show the condition more clearly than a large number. Considerable judgment must be used in selecting those soundings which best indicate the subsurface conditions.

Section VIII. METHODS OF TAKING AND LOCATING SOUNDINGS

289. Soundings Within Visible Range of Shore Stations

When the survey area is within visible range of shore stations, the soundings are located by one of the methods described in this section. The depth measurements are made by leadline, sounding pole, or fathometer.

290. Locating Soundings by Range and Angle From Shore

The position of a sounding may be located by steering the boat on a fixed range and measuring the angle from a fixed line on shore to the position of the leadsman at the instant the sounding is taken.

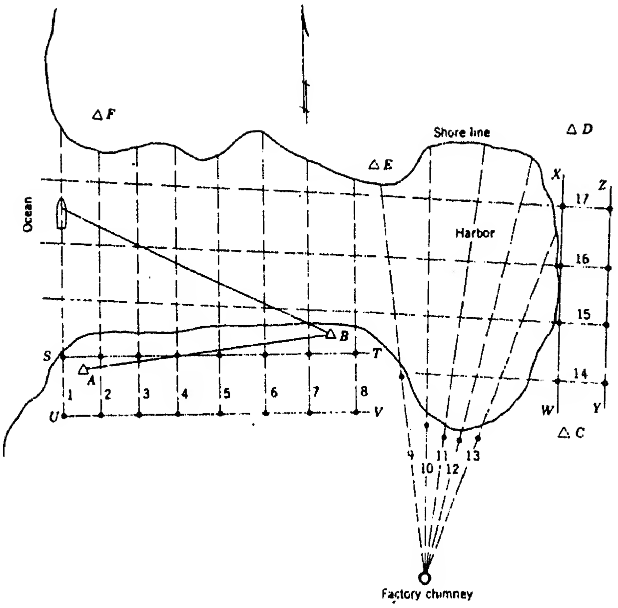


Figure 122. Soundings located by range and angle.

a. Preliminary Work. Figure 122 shows a system of ranges and control stations for locating soundings in a small harbor. Points A to F inclusive indicate triangulation stations, traverse points, or hydrographic stations which have been located by the control or topographic parties and which can be occupied by transit or sextant parties. Lines ST, UV, WX and YZ are established at convenient locations by a shore party. Stakes are placed as indicated by the black dots in the figure to provide range lines at such intervals that the underwater conditions will be correctly indicated by the soundings taken along them. It may not be always possible to set the range stakes along parallel lines as indicated in the figure but the stakes can be located more rapidly if this can be done. In the inner harbor, where waterfront buildings may make it impossible to set two ranges in a line back from the shore, single range flags are erected at points on the shore or on wharves or roofs in such locations that the flags can be lined up with the distant factory chimney to form a system of radiating range lines. Ranges 14 to 17 inclusive are set to provide cross lines of soundings to check the accuracy of the work and the thoroughness of the coverage. All range stakes, such intermediate transit stations as may be needed, the shoreline, and the topography and physical features are located by the topographic party usually on a planetable sheet on which the main control stations have been plotted. This provides a map of the area on which the soundings can be plotted when the field measurements have been completed.

b. Recommended Method. A shore party of one or two men will set range flags at the stakes marking the first few lines of ranges. This party will move the range flags to other lines as the soundings are completed on one line and the work proceeds to other ranges. The watches carried by the boat party, transit party, and tide-gage observer are synchronized and a system of flag signals is agreed upon for indicating the first and last soundings on each range and all intermediate soundings which are to be located by angles. The transit party, consisting of an instrumentman and possibly a recorder, then proceeds to station B from which good intersections with range lines 1 to 5 can be obtained. The boat party moves to range 1 and proceeds, under power or by rowing, along the range line, taking soundings at intervals. The use of portable radiotelephones will assist in keeping the sounding boat on range, in shifting the positions of shore units, and in properly coordinating the operations of all units.

(1) *Transit party operations.* The instrumentman sets his transit over station B and sights station A with the vernier reading zero. He then loosens the upper motion and turns the telescope slowly to the right, keeping the leadsman in line. Shortly before a sounding is to be taken, a flag will be raised in the boat. This is dropped at the instant the depth measurement is made. When the flag drops, the instrumentman stops the motion of his telescope, notes the time and the color of the flag, and reads the angle. Working alone, an instrumentman can read and record about two angles per minute. With a recorder, the speed can be doubled or tripled. When the sights are short, small errors in the angles will have little effect upon the plotted positions. Under these conditions it is

Cutler Harbor ~ Sounding Locations ~ 2 August 19					
No.	Range	Time	Flag	Angle	Remarks
⊼ @	ΔB	B.S. 0° 00' on ΔA			⊼ Sgt. L.D. Smith
					Berger # 12570
1	1	8h 15m 30s	R	15°30'	Angles to boat under command
2		17 12	W	18 15	of Lt J.M. Jones
3		18 51	W	21 50	
4		20 35	B	24 10	
5		22 10	W	27 00	
6		23 56	W	30 05	
7		25 38	B	33 20	
8	1	End 8 27 11	R	36 00	
9	2	Begin 8 30 03	R	40 10	
10		?	W	?	Tugboat on line
11		8 32 58	W	35 25	
12		34 27	B	32 10	
					J.M. Jones

Figure 123. Transit notes—sounding locations.

customary to leave the upper motion unclamped, setting on the leadsman by hand instead of using the slow-motion screw, and estimating the angle to the nearest 5 minutes instead of using the vernier. For long sights vernier readings to the nearest minute may be necessary. The notes (fig. 123) show the sounding number, the range, the time, the color of the flag, and the angle. By the time the sounding boat has reached the northerly end of range 5 (fig. 122), the intersection angle between the transit line and the range is becoming quite small and a poor intersection would be obtained at the northerly end of range 6. The transit is therefore moved to another station, for example, *A*, for observations on the next few ranges.

(2) *Boat-party operations.* The boat party will consist of the chief of party who will direct the operation and who may serve as helmsman to keep the boat on range, a leadsman, a recorder, one or more oarsmen if the boat is not powered, and in congested waters, a lookout. The leadsman stands in the bow or in a *sounding chair* (also known as *the chains*) rigged out over the water abeam of the boat (fig. 124). In the figure two boats have been lashed together with cross timbers to form a relatively stable catamaran. The

leadsman swings the lead and casts it forward so that, as the boat proceeds on its course, the lead will reach the bottom directly below him. On the larger boats, the sounding chair is high enough above the water so that the leadsman can make a long cast forward and the boat may proceed at a moderate speed. In small craft, the sounding chair cannot be far above the water and the speed of the boat must be slow to assure vertical casts in moderately deep water. The leadsman takes each sounding as directed by the party chief. A colored flag is raised shortly before each fixed sounding as a signal to the transit party and then dropped as the sounding is taken. Notes taken by the recorder (fig. 125) show the sounding number, the range, the time, the color of the flag, and the depth. Columns are provided for tide heights and the soundings reduced to datum. These are filled in at the close of the day when the record of the tide-gage observer is available. Usually a flag is shown for a transit fix at about every third or fourth sounding, the intermediate soundings being interpolated between fixed soundings in accordance with the time intervals since the boat is being rowed or operated at practically constant speed. Note that only those soundings are numbered where a flag is shown. The total numbers indicated by the boat and transit parties should check at the end of the day. Should the instrumentman fail to observe one or more flagged soundings this will be evident from a discrepancy of numbers and the missing soundings can be detected from the times and the recorded colors of flags.

c. Alternate Method. The angle on shore may be measured with a sextant instead of with a transit if the two shore stations are nearly at water level so that the angle observed is substantially horizontal. Because of its large field of view and the manner in which it is turned in azimuth, it is even more convenient for this

Figure 124. Sounding chair on catamaran.

purpose than the transit. Another procedure which is sometimes used is to have the instrumentman in the boat where he measures a sextant angle between the range and the shore station. This has the advantage of keeping the instrumentman under the direct supervision of the party chief in the boat but it increases the time spent in plotting. Where the angles are turned from a shore station, a large number are plotted from one position of the protractor; when measured from the boat, if an ordinary protractor is used for plotting, each angle must be plotted on tracing paper and the paper slid along the plotted range line until one side of the angle passes through the shore station and the other lies along the range. The position of the sounding is then pricked through the tracing paper to the map below.

291. Locating Soundings by Range and Time Interval

If a boat proceeds along a range of known length at a constant speed and soundings are taken at regular intervals during the time required to move from one end of the range to the other, the soundings can be located by proportioning the time intervals. For example, the length of range 1 (fig. 122) can be scaled from the shoreline survey map. Assume this distance to be 3,400 feet. If buoys are placed on range at 200 feet from each shore, there should be room for maneuvering shoreward of each buoy so that the boat may be brought up to speed before passing the first buoy and turned, after

Cutler Harbor ~ Soundings ~ 2 August 19--

No.	Range	Time	Flag	Sounding (Fathoms)	Tide	Reduced Sounding		Remarks:
1	1	Begin 8h 15m 30s	R	2.6	0.8	1.8		Lt. J.M. Jones C.O.
	Running Northerly	16 02		2.8		2.0		Cpl. T.M. Wells, Recorder
		16 35		3.0		2.2		Pfc. J.H. Shea, Leadsman
2		17 12	W	3.4		2.6		Pfc. T.A. Gifford, Gage
		17 43		3.6		2.8		Sgt. L.D. Smith at ⊼
		18 12		3.8		3.0		on △B.
3		18 51	W	4.0		3.2		
		19 22		4.1		3.3		
		19 56		4.5		3.7		
4		20 35	B	4.8		4.0		
8	1	8 25 38	R	3.6	0.8	2.8		
9	2	8 30 03	R	4.0	0.9	3.1		
	Running Southerly	30 36		4.2		3.3		
		31 08		4.3		3.4		
10		31 44	W	4.6		3.7		
								J.M. Jones

Figure 125. Boat notes—soundings.

passing the one near the far shore, without running aground. If soundings are taken at each buoy and at 30-second intervals between them and it requires 5 minutes to traverse the distance of 3,000 feet between them, there will be a total of 11 soundings and 10 spaces between them. Each spacing will then be 300 feet. This method is also employed in offshore work where the distance between the two ends of a run along a definite course can be fixed by astronomic observations, radio-acoustic ranging, or other methods.

292. Locating Soundings by Intersecting Ranges

Reference to figure 122 will show that if a sounding is taken at the instant that a boat, proceeding along range 10, is lined up with range 14, the sounding can be plotted at the intersection of the two range lines. This method of location can be used where there are many bends in the shoreline or, on a river, where a bridge span permits the erection or range flags on the two trusses or rails at approximately right angles to ranges placed along the bank. It is also used for taking soundings within cofferdams for bridge piers or other subaqueous structures where the intersecting lines of timber bracing provide definite locations for depth measurements.

293. Locating Soundings by Two Angles From Shore

In many areas where the shores are precip-

itous or heavily forested it may be difficult or impossible to establish ranges on shore. There may also be times when river or tidal currents are too strong to permit keeping the boat accurately on established ranges. Under such conditions soundings may be located by taking two transit or sextant angles simultaneously from points on shore. For example, two simultaneous transit angles from the line *AB* (fig. 122), turned from *A* and *B* respectively, would suffice for locating the sounding boat at almost any point in the area between and adjacent to them without the need for establishing the ranges. The transit notes would be similar to those shown in figure 123 except that the column for the range would be omitted. It is important that the two transit lines to the boat should intersect at as near a right angle as possible since lines intersecting at very sharp or very flat angles define the point of intersection poorly. It is also important that the boat, when not moving on a fixed range, be steered on a definite compass course or toward some definite object on the far shore to prevent the soundings from being taken in a hit and miss fashion. Unless this is done the soundings may be too closely spaced in parts of the area while other sections are inadequately covered.

294. Locating Soundings by Two Angles From the Boat

a. Method. One of the commonest methods of locating soundings is that of taking two simultaneous sextant angles from the boat to three previously located stations. These may be either shore signals or buoy stations. The method is an application of the three-point problem used in plane-table surveys (TM 5–232). In figure 126, if angle α is measured between the hydrographic signals *Bat* and *Doe* at the same time that angle β is measured between signals *Doe* and *Sow* (para. 286 for system of naming signals) there is, unless the boat lies at a point on the circle passing through the three stations, or the two men measuring their respective angles are not close together, only one position of the boat from which the three rays representing the sides of the angle measured will pass through the three hydrographic stations. If the boat is on the circle passing through the three stations its position is indeterminate. The accuracy of the fix depends upon the relative positions of the signals. The fix is strong if the three stations are nearly in line or if the middle station is closer to the boat than the other two. Small angles generally indicate a weak fix although a good location may be obtained when the angle between two is small but the stations are far apart and the second angle is relatively large. The chief of party must show good judgment in selecting the stations used for fixes. For methods of plotting the positions see TM 5–232. This method has the advantage that all personnel are in the boat and under the immediate direction of the unit commander. It has the further advantage that, in craft the size of a launch or larger, the soundings can be plotted on a boat sheet affixed to a plotting board as the measurements are made, so that the adequacy of coverage can be determined and additional soundings taken at once where too few measurements have been made. Necessity for plotting instruments in the boat may be eliminated by the use of a previously prepared sextant arc-sheet. This method has the same advantage
has over stadia measurements ...
ting can be done while the work is in

b. Personnel and Operation. The boat party consists of the party chief who usually reads the left-hand sextant angle and plots the position on the boat sheet, a right-hand angle man, a recorder, a helmsman or coxswain, a leadsman, and such additional personnel as may be required to operate the boat. The duties of the personnel are discussed in paragraph 298. The notes (fig. 127) must include additional columns for entering the names of the signals sighted and the values of the sextant angles. The boat is steered on definite courses or ranges and sextant fixes are obtained periodically, the intermediate soundings being interpolated on the boat sheet in accordance with the time inter-

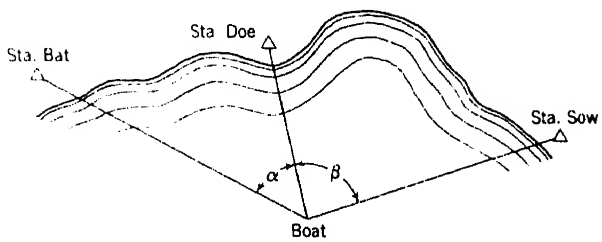


Figure 126. Sextant angles from boat.

	Eastport Harbor ~ Soundings ~ 3 August 19__							
Position Number	Time h m s	Sounding Feet	Tide Feet	Reduced Sounding Feet	Bottom	Compass Course	Angles and Ranges	Remarks
								Lt. T. C. Mann CO
								Left Sextant
$\angle$25	09 58 15	7.5	4.0	3.5	Rky	110°	$\angle$25 Bat. 48°10'	Sgt. A. L. Ball
	59 00	8.0		4.0			Doe	Right Sextant
	15	9.0		5.0			Sow 87 12	Pfc. E. White
	30	9.5		5.5				Recorder
	45	10.0		6.0				Pfc. L. Brown
$\angle$26	10 00 00	6.5		2.5	Bk. S		$\angle$26 Bat. 42 15	Leadsman
	20	15.0		11.0			Doe	Pfc. O. M. Ray
	40	19.5		15.5			Sow 84 24	Gage
	01 00	24.0		20.0				Verified by leadsman
								T. C. Mann

Figure 127. Sounding notes—sextant fix.

vals. Figure 128 shows sextant angles being taken from a launch.

295. Locating Soundings by Stadia or Planetable

Where the water is shallow a stadia rod may be used to provide a measure of both the horizontal distance and the depth. The lower end of the rod is placed on the bottom alongside the boat, the depth recorded, and the stadia distance and azimuth read from a transit on shore. The transit station should be as near the water level as possible so that it will not be necessary to apply a correction for vertical angle to the rod reading. If the boat is rowed along a range and the transit is placed on the range where it meets the shore, the need for an azimuth angle is eliminated and it is only necessary to record the distance out along the range. This method may also be used to locate soundings on lakes or rivers when they are taken through holes cut in the ice the rod being held on the ice alongside each hole and the azimuths and distances measured with a transit on shore or set over a control point on the ice. In extremely cold weather it is usually more convenient to locate such soundings at the intersection of range lines (para. 292), thus avoiding the necessity of having a transitman occupy an exposed station for a protracted period. In instances where a planetable has been used for plotting the topography along the shore, the planetable may be substituted

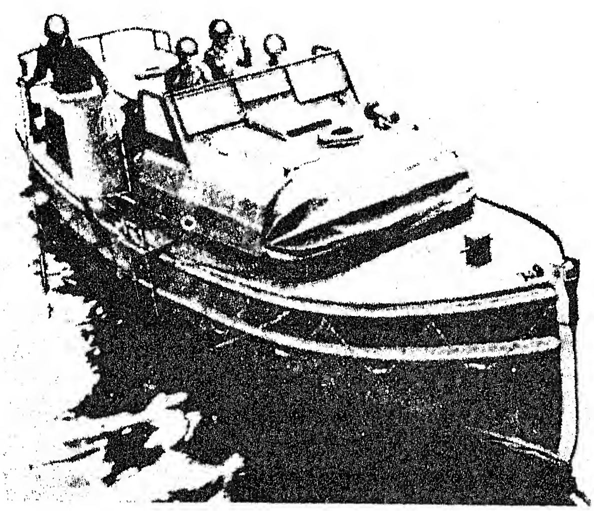


Figure 128. Sextant fix from launch.

for a transit in this method, the soundings being plotted directly on the planetable sheet.

296. Locating Soundings by Floated Wire

Soundings can be located with accuracy in narrow waterways by an adaptation of a method used by the U.S. Coast and Geodetic Survey for distance measurements in planetable traversing. Along low, flat coasts with little vegetation, heat waves tend to decrease the visibility and accuracy of stadia distances. The distances are therefore measured with a 100-meter length of stranded sounding wire, graduated at 25-meter intervals by small pieces of cloth inserted between the strands, and provided with holding toggles at each end. It is nothing more than a special form of steel tape adapted for rough work. Where a traverse side crosses a waterway, fittings, to which floats may be attached, are secured to the wire at short intervals and the distance is measured by the floated wire. Such a wire can be graduated in any desired manner, stretched across a narrow waterway or stretched between anchored buoys to form a traverse along a winding channel, and soundings can then be taken at each graduation along the wire. A somewhat similar device, the *taut-wire apparatus,* is used to measure the distances between offshore control buoys. One end of the wire is anchored at the first buoy and the wire is payed out over a registering sheave as the ship proceeds on course to the next buoy station. If soundings are to be taken along the line as the ship proceeds, their locations can be fixed from the readings of the registering sheave.

297. Methods of Fixing Positions When Beyond Visual Range of Shore Stations

The above methods of fixing the position of a sounding vessel are not applicable when the sounding vessel is offshore or when fog or darkness shroud the survey area. Methods for the location of the sounding vessel under such conditions include electronic position indicator, radar, shoran, loran, and astronomical determination of position. Engineer troops will not be expected to accomplish hydrographic surveys under conditions which necessitate the use of these methods.

298. Duties of Survey and Sounding Party

a. Chief of Party. The chief of party will—

(1) Direct all survey operations of the party and issue all necessary orders and instructions including directions to the helmsman and instructions as to the intervals between soundings, ranges, and fixes, and the shore or buoy stations to be used for sextant angles.

(2) Assume responsibility for the accuracy and adequacy of the results and the safety and care of the boat, personnel, and equipment.

(3) Take the left-hand angle where sextant locations are used and plot the soundings on the boat sheet if one is used.

(4) Train himself to note subconsciously all soundings, angles, and other data which are called out and repeated, so as to know at all times the depths being obtained and whether the values are being correctly repeated by the recorder.

(5) Immediately order vertification when any depths are measured which differ greatly from those immediately preceding (*Remarks* col, fig. 127).

(6) Approve the record of each day's work with his signature.

b. Instrumentman. The duties of an instrumentman locating soundings from a shore station are given in paragraph 290*b*(1). Where sextant angles are measured in the boat, the instrumentman normally measures the right-hand angle, supervises the work of the recorder, and oversees the sounding operation. If the boat crew is small, he may also serve as a lookout.

c. Recorder. The recorder will repeat clearly all values of depth, flags shown, sextant angles measured, control stations sighted, times, and all other data as they are called out to him and will record the values legibly in the appropriate columns of the sounding record.

d. Fathometer Attendant. In deep water surveys where the bottom is regular and the soundings are not closely spaced, the duties of the fathometer attendant and those of the recorder may be combined. In most operations it is essential that the attendant devote his undivided attention to the fathometer so that the depth of water is continuously observed and there is assurance that no shoals go unnoticed. A regular sounding interval, depending upon the scale of the chart and the speed of the boat, is ordered by the chief of party and depths must be read at this prescribed interval. The attendant must also note the shallowest and deepest soundings. These must be reported to the recorder if they differ by more than 5 percent of the depth from the general slope of the bottom between the soundings taken at regular intervals. The attendant must be familiar with the adjustments of the fathometer. If operating difficulties develop with which he cannot cope, he must notify the party chief at once.

e. Leadsman. The leadsman (fig. 129) stands in the bow of the boat or, preferably, in a sounding chair, and, when directed, casts the lead well forward so that a vertical measurement of the depth is obtained. He should call off the depth to the recorder, being very careful to enunciate clearly and to differentiate between words such as seven and eleven which might be confused. He must listen to be sure that the recorder correctly repeats the depths given. He must also keep a lookout over the surrounding water and must report immediately any shoals, discolorations, or objects, which may indicate a possible danger to the vessel. Because of the effort required in heaving the lead, the leadsman is usually relieved hourly, frequently alternating with the coxswain. He is also frequently provided with an assistant to haul in the leadline after each sounding and to arm the lead when samples of the bottom material are to be taken.

Figure 129. Casting the lead.

f. Coxswain. The coxswain steers the boat and, when an outboard engine is used, generally operates the engine and controls the speed. Larger craft, powered by inboard engines, will usually have a marine engineman. The coxswain steers on compass bearings or on ranges as directed by the party chief. He must learn to hold the boat accurately on a range and must be alert for orders to change course after a position has been fixed or when the soundings on a given range have been completed. He repeats, for verification, all orders issued to him.

g. Lookout. In many instances the leadsman, the instrumentman, or the coxswain may be directed to keep a lookout in addition to his other duties. In heavily congested waters or in waters where there are many dangers to navigation, a lookout may be stationed in as elevated a position as possible to report all hazards promptly so that proper orders can be issued to assure the safety of the vessel.

299. Fathometers

a. General Principle of Instrument. A fathometer is an instrument for measuring the depth of water by determining the time required for sound waves to travel, at known velocity, from a point on a ship near the water surface to the sea bottom and return. One-half of the elapsed time between the production of the sound on the ship and the reception of its echo from the bottom, when multiplied by the velocity of sound in sea water, gives the distance from the instrument to the bottom. A correction factor must be introduced to give the true depth since the instrument is below the waterline. In practice, the conversion of the time interval to a measurement of the depth in feet or in fathoms is performed by the instrument. While simple in principle, the design of the instrument is complex. Echo sounding is affected by a number of varying conditions including the temperature and salinity of the water, aeration, turbulence, absorption of sound waves, reflection from surfaces other than the sea bottom beneath the ship, and the type of bottom. Instruments have been developed which measure depths from a few inches to the greatest ocean depths of over 6,000 fathoms with greater accuracy, far more rapidity, and greater ease than is possible with the older methods of taking soundings.

b. Basic Elements. A fathometer is composed of three principal elements performing separate functions which, in combination, produce a depth measurement.

(1) The acoustic transmitting unit emits signals at sonic or supersonic frequencies and is provided with a suitable source of energy to operate it.

(2) The acoustic receiving unit and amplifier receives the echo and converts the acoustic energy into electrical energy.

(3) The registering device measures the time interval between emission and reception of signals converts this to units of depth, and then registers this depth either visually or on a graphical record.

The development of these instruments has been rapid during the past 30 years and improvements are continually being made. U.S. Coast and Geodetic Survey Special Publication 143, *Hydrographic Manual,* contains complete descriptions of the several instruments in use at the time of its publication.

c. Classification. Fathometers are classified with respect to type of registering devices, depth range, frequency and portability.

(1) *Registering devices.* Fathometers are provided with either a visual depth indicator or a graphic recording device. The depth at any time can be read quickly and accurately on a visual indicator, but the instrument must be observed constantly if a profile of the bottom is desired. It is most used when only occasional soundings are required. The graphic recorder provides a permanent record of the bottom profile beneath the track of the ship. Generally, instruments of both types are operated simultaneously on a ship to provide checks and to supplement one another. Figure 130 shows a fathogram of a rugged bottom with many rock pinnacles.

(2) *Depth range.* Certain fathometers are designed for accurate measurements in relatively shallow water from 0 to 20 fathoms. Others are used in moderate depths of from 20 to 100 fathoms, while still others are designed for deep water soundings. A few instruments register accurately throughout the entire depth range.

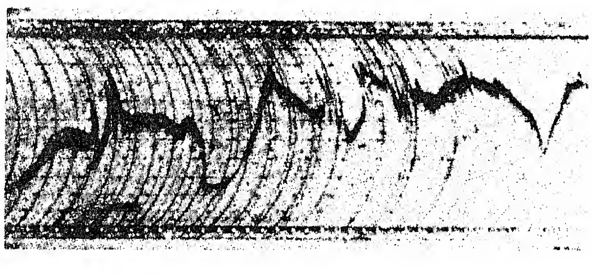


Figure 130. Fathogram of rugged bottom showing rock pinnacles.

(3) *Frequency.* The transmitter sound may be at an audible or sonic frequency or at a supersonic frequency inaudible to the human ear. Instruments with sonic frequencies are much used for navigation because the equipment is relatively simple and inexpensive. They are also useful for deep soundings because absorption is low at these frequencies. Most survey fathometers employ supersonic frequencies. Such instruments are more accurate in shoal water and are generally equally satisfactory at greater depths. Because of the short wavelength, greater detail is obtained when the sea bottom is very irregular. The sound waves can be beamed directly toward the bottom, resulting in a reduction in side echoes. Spurious sounds are also reduced in the receiving unit both because of the beaming and because most of the ship and water noises are at lower frequencies.

(4) *Portability.* The larger fathometers are permanently installed in a vessel. Other types are classed as portable or semiportable and are commonly installed in launches for shoal water surveys where they may be transferred from one vessel to another as need arises.

d. Operating Instructions. No models of the fathometer have been standardized for military issue. Reference is made to U.S. Coast and Geodetic Survey Special Publication 143, *Hydrographic Manual* for data concerning the operation of instruments in use at the time of its publication. Instruments requisitioned for military operations should be accompanied with specific instructions governing their use.

300. Sounding Poles and Leadlines

a. Sounding Poles. Soundings over extensive areas of shoal water less than 2 fathoms in depth can be measured more accurately and more easily with a sounding pole than with a leadline. The pole is usually of rounded wood, about 1½inches in diameter and 15 feet long. It is provided at each end with a metal shoe that may be weighed to aid in sinking the lower end to the bottom. Graduations in feet and half-feet are painted from the center toward each end and are numbered consecutively from the ends toward the center. In use, the pole is not raised completely from the water between soundings, but is turned alternately end for end, somewhat after the fashion of using a double canoe paddle. This increases the speed of the work, permits taking soundings close together, and is easy on the operator. Sounding poles of graduated iron rods or pipes are also used for depth measurements in high-velocity streams or tidal currents where vertical measurements with a leadline are difficult, if not impossible.

b. Leadlines. A leadline is a graduated length of sash cord or tiller rope with a lead weight attached to its lower end.

(1) *Lengths and weights.* The length and weight used on a given leadline will depend upon the depth and velocity of the water to be sounded. Leadline measurements are used for depths not exceeding 30 fathoms. An 8-pound weight is satisfactory for depths up to 7 fathoms. For greater depths, 12- to 14-pound weights are generally used and even heavier leads are used to assure vertical measurements where high current velocities are encountered. To avoid varying the tension on a line, which might introduce errors in the measurements, the same weight is always used on a given line.

(2) *Preparing the line.* The line must be carefully prepared to avoid errors from subsequent stretching or shrinking. Tiller rope is the best material for the line. This has a phosphor-bronze wire core covered with waterproofed braided cotton. The line is soaked in salt water for 24 hours. The cotton covering is then worked back, a little at a time, along the wire core until about a foot of wire protrudes for each 10 fathoms of line. The excess wire is cut off, the wire dried under considerable tension and then soaked for another 24 hours before being graduated. The lead is then attached

and the line put under tension equal to the weight of the lead. Graduations as indicated below are then bound on with waxed linen thread. Half-fathom marks are shown by a seizing of black thread on fathom-graduated lines. Even $\frac{1}{10}$ fathoms have a seizing of white thread. The odd $\frac{1}{10}$ fathoms are estimated.

Fathom-Graduated Leadlines

Fathoms	*Marks*
1, 11, 21	One strip of leather.
2, 12, 22,	Two strips of leather.
3, 13, 23	Blue bunting.
4, 14, 24	Two strips of leather secured in the middle so that two ends point upward and two downward.
5, 15, 25	White bunting.
6, 16	White cord with one knot.
7, 17	Red bunting.
8, 18	Three strips of leather.
9, 19	Yellow bunting.
10	Leather with one hole.
20	Leather with two holes.
2, 12, 22, etc	Red bunting.
4, 14, 24, etc	White bunting.
6, 16, 26, etc	Blue bunting.
8, 18, 28, etc	Yellow bunting.
10, 60, 110	One strip of leather.
20, 70, 120	Two strips of leather.
30, 80, 130	Leather with two holes.
40, 90, 140	Leather with one hole.
50	Star-shaped leather.
100	Star-shaped leather with one hole.
Odd feet	White seizing.

Marks are of such a size and are so secured that they project about 2 inches from the line. The above markings are those used by the U.S. Coast and Geodetic Survey. The line is usually graduated by placing copper tacks in the deck of the ship or along a wharf at the appropriate foot of fathom marks, stretching the wet line over these, marking it with temporary graduations, and then seizing-on the permanent markings. The spacing between the copper tacks is also used as a standard for daily comparisons to detect any shrinkage of stretching. Lines prepared as indicated above will maintain an almost constant length.

(3) *Lines for deep soundings.* Deep soundings are measured with lines of stranded wire or of piano wire wound on a reel and lowered and raised by either a hand-operated or power-driven sounding machine. As the wire leaves the reel, it passes over a registering sheave which provides a record of the number of fathoms.

(4) *Arming the lead.* The lead weight is provided with a cup-shaped depression at its lower end. This may be filled with soap or tallow and the lead *armed* for the collection of samples of the mud, sand, shell, or other material on the sea bottom.

301. Sweeps and Drags

Leadline soundings normally give satisfactory indications of the depth in waterways when the bottom is comparatively regular but, even when taken at short intervals, they cannot be relied upon to show minimum depth over rock pinnacles, sunken wrecks, or other dangers. Figure 131 shows the failure of the sounding lead at casts *A* and *B* to indicate the presence of a large boulder. At cast *C*, the lead has slipped down the sloping face of the rock pinnacle and shows a considerable depth although the pinnacle rises to within a few feet of the surface. To detect and locate such dangers, wire drags, wire sweeps, or sweep bars are used.

a. Wire Drag. The wire drag (fig. 132) consists of a horizontal bottom-wire which is supported at intervals by adjustable upright cables suspended from surface buoys. The upright cables can be lengthened or shortened to sweep at a given depth and to make adjustments for the rise and fall of the tide. They are maintained in a nearly vertical position by weights

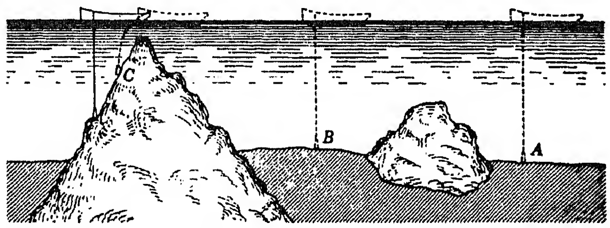


Figure 131. Dangers undetected by leadline soundings.

attached at their lower ends. The drag wire is prevented from sagging between uprights by a number of intermediate wooden floats attached directly to the wire. The drag wire, which may have a length of 5 miles (8 km), is towed by two launches (fig. 132). Obstructions are located as indicated, a buoy is placed over each obstruction, the drag is cleared, and a small sounding boat is detailed to make careful soundings in the buoyed area to determine the minimum depth over the obstruction.

b. Wire Sweep. The wire sweep is a modification of the wire drag. Buoys are placed farther apart and there is no provision for varying the depth of the wire while dragging, nor for preventing the sag of the wire between buoys. It may be placed in operation more quickly than the drag and is used chiefly in areas where few obstructions are to be expected and where the general depth of the water is considerably greater than depths required for navigation. U.S. Coast and Geodetic Survey Special Publication 118, *Construction and Operation of the Wire Drag and Sweep,* contains much data on the use of these devices. A somewhat different method of sweeping, using three echo-sounders feeding into a single recording unit, has been developed by the Chicago District, Corps of Engineers.

c. Sweep Bar. The sweep bar is used primarily for accurately determining minimum clear depths and locating obstructions and dangers to navigation such as shoals, rock pinnacles, reefs, or wrecks in confined areas such as channels, anchorage areas adjacent to dock lines, quay walls, and the like. The sweep bar consists of a heavy section of railroad rail, steel pipe or a structural steel section, supported at a predetermined depth by two vertical cables. It can be suspended from a float, catamaran or boat of suitable size. The supporting craft may be towed or self-propelled but must be highly maneuverable to maintain precise horizontal control of sweeping operations. The sweep bar is suspended so as to avoid sag or deflection and the depth can be controlled precisely from the supporting craft by adjusting the cables holding the bar. The use of the sweep bar has wide applications, ranging from a small hand sweep operated from a rowboat to locate minor obstructions in constricted areas, to specially designed craft for sweeping large channels and harbors.

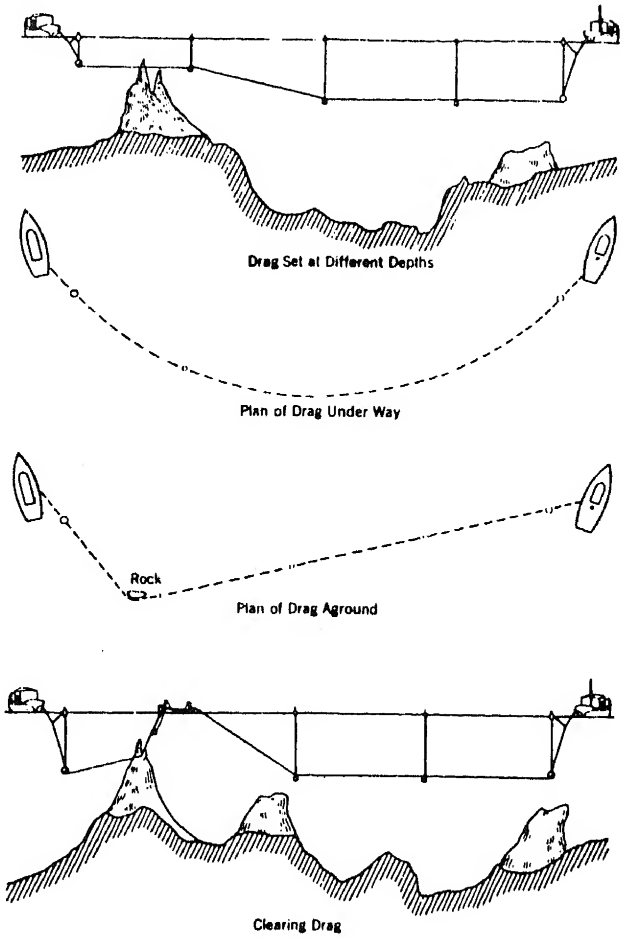


Figure 132. Diagram of wire drag in operation.

302. Boats

For deep water surveys offshore, special survey vessels up to 200 feet in length are used. These boats have accommodations for personnel and are provided with a drafting room and all necessary instruments and equipment. Smaller auxiliary vessels are used for hydrographic surveys in protected waters. Engineer boats will be used whenever available. These include the gasoline-powered bridge erection boat and the gasoline-powered utility boat. Transportation Corps boats may be available from a transportation boat company. When issue boats are not available, and it is necessary to hire sounding craft, many types can be utilized. These include dories, skiffs, dinghies, runabouts, and

whaleboats. Such craft may be provided with inboard or outboard motors or may be rowed. Outboard motors, which have a wide speed range, are preferable. Fishing craft or other workboats commonly used in a locality will generally be found to be seaworthy and more satisfactory for survey purposes than pleasure craft. These boats are used for sounding and for the transportation of working parties and materials for the triangulation, topographic, and other shore parties as well. Two small boats are often connected by crossbeams to form a catamaran (fig. 133) which provides greater stability.

303. Buoys

Survey buoys are required to control hydrographic surveys beyond the visibility of shore signals. Marker buoys are placed over shoals and other rangers, to control radiating lines of soundings, and for other purposes. Such marker buoys may consist of a 5-gallon can secured to an anchor by a light line or wire. The visibility of such a buoy is increased by lashing a pole to it with a signal flag on the upper end of the pole and a counterweight on the lower end. Survey buoys usually use from one to four 55-gallon oil drums with 2- by 4-inch lumber and 1- by 4-inch bracing for the necessary signal and counterweight framing. Figure 134 shows a typical one-barrel buoy. For identification, the name of the buoy station (para. 286) is painted on the buoy. Buoys placed near shipping lanes should be lighted at night. An inexpensive flashing light with batteries in a waterproof box can be secured to the buoy superstructure. In regions where daily tide fluctuations are negligible, an effective buoy can be made from 30 to 35 feet of spruce or cedar. Buoyancy is obtained by means of cedar blocks attached at a point one third of the distance from the bottom of the pole. The buoy is flagged and anchored at the proper depth with the tops of the cedar blocks at the approximate water surface.

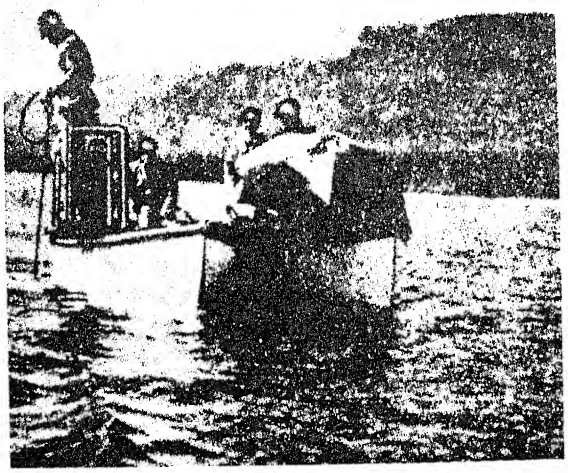


Figure 133. Soundings from catamaran.

304. Signals

For triangulation and traverse control and for shore topography the types of signals, lights, and targets commonly used for such surveys are employed (TM 5–441). When the position of a sounding boat is obtained from a sextant-fix, various types of hydrographic signals on shore, or buoys anchored in the water, are used for sights. These must be sufficiently conspicuous to be distinguished readily in the mirror of the sextant (para. 305). Many structures on shore, erected for other purposes, make satisfactory sextant sights. These may include lighthouses, chimney stacks, flagpoles, water tanks, triangulation towers, and similar prominent objects. Tripod signals used for topographic surveys can be seen from offshore if suitably dressed with bands of cloth or painted boards nailed between the tripod legs. Pylons made of light lumber and wrapped with cloth for increased visibility are useful. Whitewash applied to rocks or ledges on points of land, islands, and rocks extending above high water will make a signal which is visible for a considerable distance. Flags on poles nailed in tree tops or an unusually tall tree stripped of all limbs except a tuft at the top may be used. Where a coastline is low and flat it is sometimes necessary to erect very tall signals to provide visibility for some distance to seaward, but offshore buoys are more commonly used for control in this case. The hydrographer selects or devises signals which can be seen readily from the boats. These signals must be located at sufficiently frequent intervals, approximately 1,200 feet (370 m), along the shore to provide strong sextant fixes. For offshore

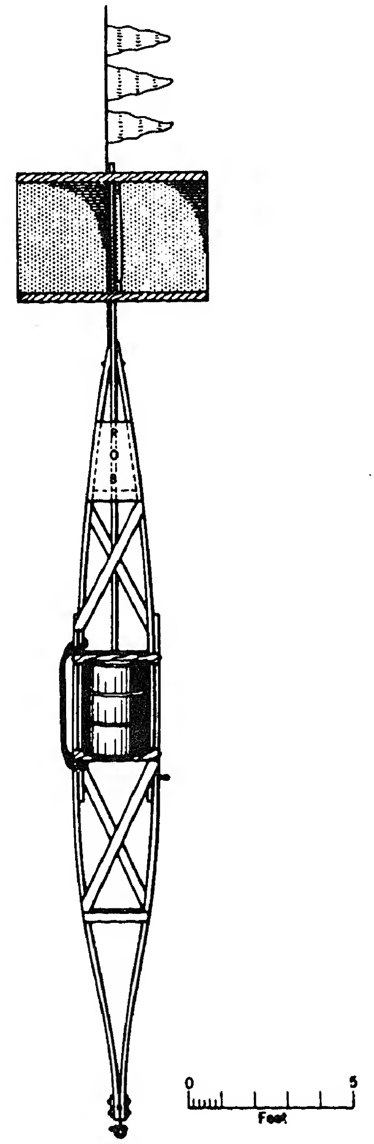


Figure 134. One-barrel survey buoy.

work the distance between signals may be several miles, but the signals must be tall or located on elevated points to be visible. It is important that the size, shape, and spacing of these signals be varied to permit easy identification. Signals which are close together and similar in size and shape may be incorrectly identified when viewed from a mile or two offshore in a small sextant mirror. Where these signals are to be used for some time, durable signal notices, printed on muslin or stencilled on a board, should be tacked to them to minimize the danger of disturbance or destruction by persons unaware of their purpose. Signals and buoys are also used to mark range lines so that a sounding boat may be lined up with two signals on shore, with a signal and a buoy, or with two buoys, and then be steered on this range. Flagged range poles are frequently used for this purpose since they must be moved from range to range at frequent intervals.

305. The Sextanr

The sextant is a portable instrument for measuring the angle between two objects. Unlike the transit, it does not require a stable support and hence is eminently suited for angle measurements from a boat or ship. The angle is measured in the plane passing through the two objects and the instrument, hence the angle is only horizontal or vertical when these three points are in a horizontal or vertical plane.

a. Description. The sextant (fig. 135) consists of a rigid metal frame *ABI*, usually of brass or other alloy. The underside of the frame has a wooden handle *D* and three short legs for supporting the instrument when it is laid on a table. The arc or limb *AB* has its center at a pivot *I*. This arc carries the graduations on an inlaid silver strip. Although the arc shows graduations up to 150°, the spaces are actually ½° in width since, in operation, the index arm *IE* moves over an arc equal to one-half the measured angle. This index arm is pivoted at the center *I*. Its other end is supported by a grooved guide which slides along the arc and which can be held in any desired position by the clamp *C*. Slow motion of the arm is provided by the tangent screw *T*. The arm carries a vernier *V* and a magnifying glass *S* for reading the angle. It also carries an index mirror at *I*, set with its plane perpendicular to the plane of the frame. A second mirror *H*, known as the horizon glass, is attached to the frame in a plane perpendicular to it. These mirrors are so set that they are parallel when the vernier of the index arm reads zero on the arc. The upper half of the horizon glass is unsilvered, so that an object may be viewed through it, while the lower half is a mirror. A telescope *F* is screwed into a ring *R* which may be raised or lowered slightly with respect to the plane of the frame and clamped

at the desired height. Several telescopes of differing magnifying powers and a plain tube are usually provided with the instrument. In locating soundings, it is often convenient to remove the telescope, the ring itself serving as an eyepiece. At K and L there are a number of small colored glasses of varying depths of color which are pivoted to the frame. These may be swung into position to protect the eye of the observer when sighting on the sun.

b. Use of Sextant. In figure 136, AB represents the arc, Z is the position of the index arm for zero readings, E is the position of the eyepiece of the telescope, H and I represent the horizon glass and the index mirror respectively, and C and D are the objects between which the angle CED is to be measured. To measure this angle—

(1) Hold the instrument by the handle with the right hand so that the plane of the frame lies in the plane passing through the two objects and the observer.

(2) View the object at C through the telescope E, looking through the upper transparent portion of the horizon glass H.

(3) With the left hand, move the index arm slowly along the arc, moving the index glass I with the arm, until a position V is reached where rays of light from the object at D are reflected by the index mirror I to the lower half

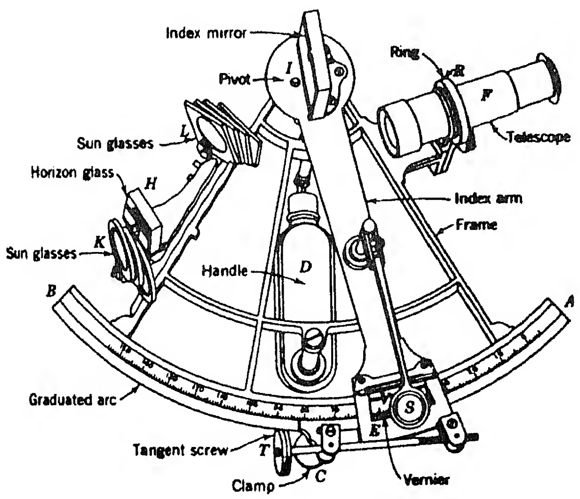


Figure 135. The sextant.

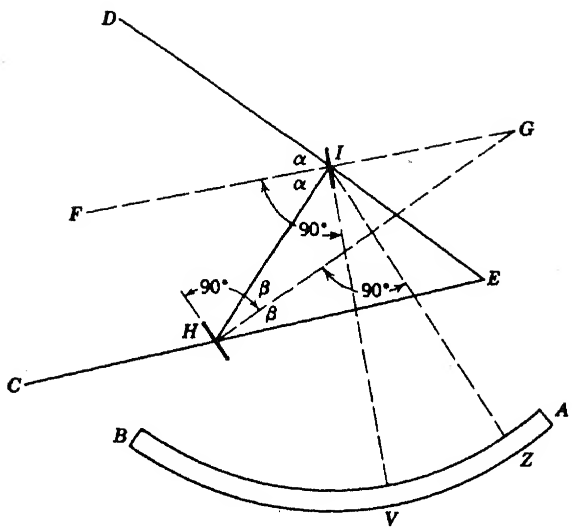


Figure 136. Principles of the sextant.

of the horizon mirror H and then to the telescope E.

(4) When this position is reached, the image of C will appear directly above and in line with the reflected image of D.

(5) Clamp the index arm at this point V and perfect the coincidence of the images by manipulating the tangent screw.

(6) Read the angle ZV. This angle is actually equal to one-half of the required angle CED. Since half-degree spaces are shown as whole degrees on the arc, the reading ZV gives the value of the angle CED.

c. Theoretical Principle. There is only one position of the index arm (fig. 136) where the image of C and the reflected image of D will coincide. Once this position has been reached, any further movement of the arm, either forward or backward, will cause the reflected image to move off the horizon glass. Since half-degree spaces on the arc ZV are marked as full degrees, it is only necessary to show that angle ZIV equals one-half of angle CED to prove that the arc reading gives a true measure of this angle. When a ray of light is reflected from a plane mirror the angles of incidence and reflection are equal. Hence, the two angles

marked α are equal and the two angles marked β are equal. In triangle *IHE*

$$\text{Angle } E = DIH - IHE = 2\alpha - 2\beta = 2(\alpha - \beta).$$

In triangle *IGH*

$$\text{Angle } G = FIH - IHG = \alpha - \beta.$$

Therefore angle *E* equals twice angle *G*. But, since *IV* is perpendicular to *FG*, and *IZ* is perpendicular to *HG*, angle *G* equals angle *VIZ* which is equal to the arc *VZ*. Thus the arc reading correctly measures the angle.

306. Other Instruments and Items of Equipment

Agencies which devote much time to sounding operations, such as the U.S. Coast and Geodetic Survey, employ numerous other items of equipment and other instruments. Some of these are mentioned or described briefly in connection with the discussion of methods of locating soundings given in paragraphs 289 through 298. Full descriptions are found in U.S. Coast and Geodetic Survey Special Publication 143, *Hydrographic Manual.*

Section X. BEACH SURVEYS

307. Introduction

The most common purpose of a beach survey is to provide preliminary data for planning and construction of piers, docks, and other harbor facilities. Another main purpose also may be to obtain data useful for landing supplies, equipment, and personnel directly on the beach from landing craft or amphibious vehicles.

308. Control Methods

Control methods adopted for this type of survey must be precise in nature. The following methods normally will be the most satisfactory.

a. Range and Distance. Establish by transit and tape a series of accurately located ranges on shore, spaced at intervals suited to the scale and extent of the survey area. Ranges should be oriented normal to the beach line, and numbered and marked by stakes or suitable markers. Positions are determined by recording range number and distance from front range marker. The distance is measured either by a tag line streamed from the stern of the sounding boat or by stadia readings by an instrumentman ashore observing on a rod in the boat.

b. Two Range Method. This method is only adaptable where two sets of ranges nearby normal to each other can be established. This method eliminated the need for observing distances, but requires more survey work.

c. Two Transit Method. The position of the sounding boat or pole will be determined by angle observations from two transits on shore set up over points previously positioned. This method is precise, but notes are slow and elaborate in plotting.

d. Sextant Fix. This method is described in paragraph 305. This method normally is not adopted to beach surveys unless the area to be surveyed is very extensive.

309. Specifications

a. Hydrography. Depth of water on this type of survey is commonly measured by a sounding pole or lead line. Pressure gages or recording echo sounders may be employed where practicable depending on the extent of the survey area. In the case of a very soft bottom the sounding pole or lead line should be provided with a disk to prevent too much penetration.

b. Tide. The tide should be recorded continuously while survey is in progress and should be extended to cover 1 lunar month (29 days), if practicable. One or more permanent beach marks should be established and the elevation determined relative to an even footmark on the staff gage.

c. Current. The current measurements should be taken to measure general coastal current and also maximum ebb and flow.

d. Trafficability. The suitability of the beach and environs for landing and operation of vehicles should be determined by inspection and penetration tests. A detailed report on this subject should be prepared and included with survey data.

e. Approaches. Where the beach is to be used for supplies and personnel landings the sea approach to the beach becomes an essential part of the survey. Surf conditions, beach gradient, and bottom characteristics should be the main considerations.

CHAPTER 6
SHORE-SHIP TRIANGULATION

Section I. INTRODUCTION

310. Surveying Difficult Terrain

Some areas to be surveyed have interiors that are difficult to reach or terrain that prohibits the economical use of conventional triangulation or traverse. In such areas, it is necessary to use some method in which points offshore can be fixed and utilized for the establishment of a control survey (shore-ship triangulation); or some method in which a central unoccupied point can be observed from a series of points arranged circumferentially (umbrella traverse).

311. Shore-Ship Method of Triangulation

Recent test and field applications have indicated that by interweaving a series of simultaneous observations between points on shore and a signal mounted on a craft or ship offshore, lengths and azimuths, and consequently positions, can be carried along a coastline economically, and with sufficient precision to meet existing mapping specifications. This method is referred to as the *shore-ship* method of triangulation.

312. Principles Involved in the Shore-Ship Method

a. Computed Through Overlapping Triangles. The principle involved in the shore-ship method is based on the trigonometric functions of an angle. The method involves the extension of a chain of computed lengths and measured azimuths starting and ending with a known measured or computed length and azimuth. In essence, it is similar to a traverse, except that the distances are computed through overlapping triangles rather than measured on the ground by conventional methods.

b. Chain of Intervisible Stations. Since azimuths may be carried forward along a chain of intervisible shore stations merely by backsighting on the initial station and turning the angle to the next shore station in the direction of progress, it remains for the shore-ship observations only to furnish the required lengths in order to reduce the chain to the conditions of a traverse. By establishing measured bases and astronomic azimuths at specified intervals along the course, a means is provided for checking and adjusting the shore-ship triangulation for both distance and azimuth.

313. Hypothetical Situation

The following hypothetical situation illustrates the principles involved in the shore-ship observations and computations (fig. 137). Let it be assumed that the length MUD-TOP is known and that no other lines joining the triangulation stations are known. It is then desired that the length TOP-SIN be determined. (BAG, ROT, GUM, and TON are inaccessible, flagged stations.)

314. Procedure

a. Three observers must occupy stations MUD, TOP, and SIN simultaneously. With their telescopes in the direct position, each observer makes the following observations:

MUD (Tel direct)	*TOP (Tel direct)*	*SIN (Tel direct)*
TOP	SIN	TOP
BAG	BAG	BAG
GUM	GUM	GUM
ROT	ROT	ROT
TON	TON	TON
TOP	SIN	TOP

①, Figure 137 is the general position scheme for all stations. ②, ③, and ④, Figure 137 are individual schemes showing the observations made from each station. ⑤, Figure 137 is a computation plan of the occupied stations and their observations on one of the unoccupied,

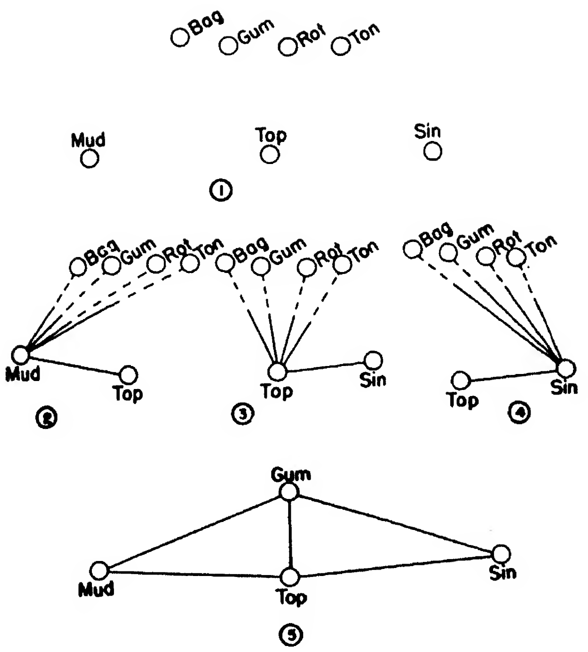


Figure 137. Example of observations in shore-trip triangulation.

flagged stations. The computation of the length TOP-SIN, based on the length MUD-TOP, could be made by using a figure of two adjacent triangles with any one of the four unoccupied intersection stations as the common vertex, in the manner illustrated in ⑤, figure 137. For increased accuracy in determining the length TOP-SIN, it would be desirable to make four computations, all in the same manner, each time using such angles as would include each of the intersection stations in order. If, for ⑤, figure 137, we introduce symbols to indicate angles and lengths instead of station names, the same computation sketch could be used for all four computations. In figure 138 the computer has a form in which he can substitute the station names as required.

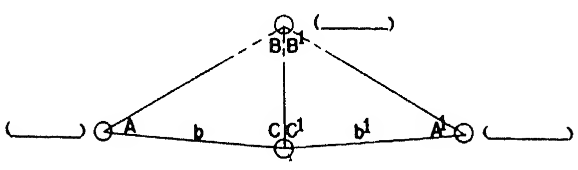


Figure 138. Example of form to aid in compiling shore-ship positions.

b. By laws of sines, the following formula will provide the required length: (Given: Side b, angles A, C, C', A'. Required: b')

$$b' = \frac{b \sin A \sin (A' \text{ plus } C')}{\sin A' \sin (A \text{ plus } C)}$$

The computation may better be accomplished by setting down the preceding formula on a form. The alphabetical symbols are the same as those in figure 138.

	Set number I	II	III
	° ′ ″	° ′ ″	° ′ ″
A angle at ( )	____	____	____
C angle at ( )	____	____	____
sum = (180° — B)	____	____	____
B	____	____	____
A' angle at ( )	____	____	____
C' angle at ( )	____	____	____
sum = (180° — B')	____	____	____
B'	____	____	____
log sin A	____	____	____
log sin B'	____	____	____
log csc A'	____	____	____
log csc B	____	____	____
sum = log X	____	____	____
mean log X	____		
log B	____		
sum = log b'	____		
b'	____	meters	
Unadjusted length ( ) to ( )		____	meters
Correction		____	
Adjusted length ( ) to ( )		____	meters

The correction is obtained from the length adjustment which is discussed in paragraphs 342 through 345.

c. In the hypothetical case concerning ① to ⑤, figure 137 inclusive, the inaccessible stations, BAG, GUM, ROT, and TON can be considered as the positions, at different times, of a signal mounted on a drifting ship. If, at these particular moments, observers at three successive shore stations were actuated by a single command "Mark!," heard by all three observers, and simultaneously had their instrument cross-wires on the signal, the group effect would be to stop all motion of the signal completely. If, instead of one pointing, furnishing information for only one computation of the length of the new line, 16 pointings were made, the 16 separately determined values for the new line would provide sufficient material for the rejection of blunders in pointings and readings. The net mean of the retained values would then provide a determination the value of which would be based upon a great number of readings, any one of which is subject to only slight errors, these errors, in turn, being of compensating nature.

315. Extension

To illustrate further the method of carrying distances through a chain of intervisible shore stations, the scheme shown in ⑤, figure 137 could be extended by a system of overlapping triangles which could be solved by the law of sines. Figure 139 shows a hypothetical extension to ⑤, figure 137 observing the angles between shore stations, and calculating the distances (by means) at the observations to the successive ship positions. Computing the traverse by this method for extending surveys it remains only to set up an organization, operating procedures, and specifications to assure achievement of the standard of accuracy desired.

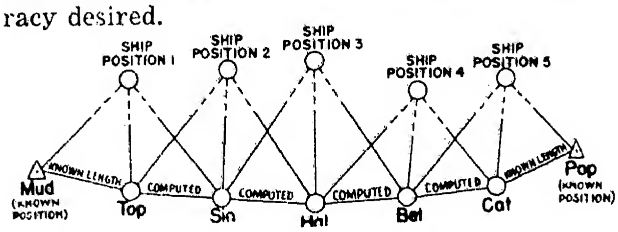


Figure 139. Hypothetical extension to ⑤, figure 137.

316. Additional Equipment

An extensive shore-ship project requires the following:

a. Craft and Signal Mark. An LST type craft or equivalent is suitable as a target ship. LCM type craft have been found to be too unstable as target ships because of the ever present wave action encountered offshore. An effective mark is provided by a pyramidal type signal about 4 feet at its base and mounted on a frame which projects the signal about 35 feet above the waterline.

b. Transportation Facilities For Shore Parties. Where overland communication between shore stations is impracticable, shore parties can best be moved by LCM type vessels. Three such craft, mothered by the LST target ship, are necessary for quick movement of the shore parties.

c. Radio Communications Equipment. Seven radio sets are necessary to provide the essential voice communications between the target ship and the shore parties. The seven sets are used as follows:

(1) One set on the target ship.
(2) One set to each of five shore parties.
(3) One set in reserve.

Section II. FIELD RECONNAISSANCE

317. Teams

The field reconnaissance team should be composed of about six men including the chief of party. Native laborers may be needed to assist where considerable clearing is required.

318. Responsibilities

The reconnaissance team is responsible for—

a. Selection of shore stations; and must insure that all adjacent shore stations are—

(1) Mutually intervisible.
(2) Between 1 and 5 miles (1.6 to 8.0 km.) apart. (The optimum distance for rapid and accurate progress is between 3 and 4 miles or about 5 km.)
(3) Marked and that a suitable signal is erected over the mark.
(4) On ground at least 15 feet (about 5 m.) above sea level.

b. Horizontal-position ties between occupied stations.

c. Selection of sites for starting, closing, and intermediate bases. (Intermediate bases are required at intervals of not less than 30 nor more than 50 miles (50 to 80 km.). Each base may be of the broken variety of not more than four breaks, or be a straight base, but must not be less than 1 mile (about 1.5 km.) in length. It is usually uneconomical to establish an intermediate base longer than 3 miles or 5 km.)

d. Clearing lines of sight to adjacent stations and toward the sea.

e. Construction of towers where necessary.

f. Location of picture points, to be pricked, described, and sketched at the site.

g. Clearing underwater obstructions which might interfere with the proper operation of signal craft.

h. Preparation of a schematic diagram, upon the completion of the reconnaissance, to include—

(1) A map plan of the signal craft and shore stations in position for the proposed operation.

(2) The approximate length and terrain conditions for each of the bases.

(3) A brief description of each of the shore stations together with the landing problems at each point.

(4) A tabulated list of the picture points together with the descriptions and the pricked photographs.

Section III. SHORE PARTIES AND TARGET SHIP

319. Shore Observing Parties

Four or preferably five shore instrument parties should be used. Normally, three of the parties are engaged in making simultaneous observations on the target craft. The remaining party or parties permit use of a shuttle arrangement to prevent loss of observing time. Occasionally, four parties may observe simultaneously to establish an additional horizontal picture point. The shore instrument parties should consist of an instrumentman, a recorder, and a radio operator.

320. Ship Party

Party personnel on board the target craft should include at least two computers, a radio repairman, a medical NCO, a boat NCO (LCM), two boat mechanics, a radio operator, and the officer in charge who will act as coordinator.

321. Classification Party

Simultaneously with the shore-ship work, a two-man (or more) classification team should be engaged in obtaining the classification data required.

322. Additional Personnel

If sufficient personnel are available, an additional five-man party should be included to operate as an astronomic azimuth and base-line team.

323. Communication

All shore observing parties are equipped with radios. While in position on shore, the shore parties will use as their radio call-sign the name of the station occupied. A shore station will be named for a local terrain feature, as is normally done for triangulation stations. Call-sign for the target ship will be CRAFT. Station CRAFT will act as the clearing station for all communications to and from shore stations. Clearance with area signal officer must be obtained.

324. Positioning of Shore Observing Parties and Target Ship

Positioning of shore parties and target ship is illustrated by figure 138. In the sketch, the numbered symbols indicate the mean position of the target ship for each set of observations, while the shore stations are shown as abbreviated names associated with a local terrain feature.

325. Procedure

The five shore observing parties station themselves at MUD, TOP, SIN, HAL, and BET. The target ship moves into position "1", about $1\frac{1}{2}$ times the mean distance of MUD-TOP and TOP-SIN and opposite the midpoint of TOP. The most efficient method of positioning the craft is by reports from the two outside stations in the set of three stations which are to do the observing. In arriving at position "1" (fig. 139) the ship would receive reports only from stations MUD and SIN. Procedure is as follows:

a. The ship positions itself far enough offshore to approach the desired position while traveling perpendicularly toward shore. In this manner the bow of the ship will be *facing* toward shore in the final position, as desired, since the signal target normally is fixed in the vicinity of the bow.

b. The survey officer in charge, using a map at the best scale available which shows the locations of the shore stations and the desired position of the ship, determines, by means of a pro-

tractor, the angles at MUD and SIN which will fix the ship's desired position.

c. Instrument parties at MUD and SIN initial on adjacent shore station and track the target ship through the telescope.

d. At request from the survey officer in charge on the craft, MUD and SIN report their angles to the ship (to the nearest degree). The craft can then be positioned by plotting the reported angles on the map. When the desired position is reached, the craft stops and the shore observations are then made and completed as described in paragraphs 326 through 335. The party at MUD then moves forward to the next station to be occupied.

e. The target ship now moves to position "2" about 1½ times the mean distance of TOP-SIN and SIN-HAL and opposite the midpoint of SIN. The ship is positioned as described above, using the reported angles from the two outside stations TOP and HAL. The shore observations are made as required and the ship proceeds to the next ship position. Party at TOP moves to the next unoccupied station.

f. The ship moves on to successive ship positions when the required shore observations are completed, while shore parties continue to leapfrog in the direction of progress after completing observations at the successive shore stations. Progress continues until all required ties are made.

Section IV. OBSERVATION PROCEDURE

326. Observation

The observations at each shore station are made in two parts. The first part consists of measuring the angle from the preceding adjacent shore station to the following adjacent shore station. This angle is called the *traverse angle*. (The observers of terminal stations in the shore-ship net as at MUD and POP have no *traverse angle* to turn.) The second part of the observations consists of measuring the angles from one of the adjacent shore stations to the signal mounted on the target ship in each of its positions. Following are the specifications and procedures for each of the parts.

327. Part I

This part of the observations should be executed at any time which will not interfere with, or delay, the program for the simultaneous observations of the pertinent shore observers on the signal craft. Four positions of the circle (1-second theodolite) will be observed on the directions; the line to the preceding shore station and the line to the following adjacent shore station. For example the observer at TOP makes the observations for the angle TOP-MUD-SIN. Observers need not close the horizon on the initial station in observing the *traverse angle*. Rejection limit of deviation, from the mean, of any one position is plus or minus 6 seconds.

328. Part 2

At each shore station, four groups of observations are made from an adjacent shore station to the signal on the craft, each pointing on the signal being made from three adjacent instruments, the simultaneous timing being maintained by radio command from the craft's radio. The telescope position for each of the four groups is alternately direct, reverse, direct, reverse. Each group is composed of first, a pointing to the most convenient adjacent shore station (preferably to the adjacent station which will permit the measurement of the acute angle rather than the obtuse angle between the next adjacent shore station and the ship); second, third, fourth, and fifth, pointings to the craft's signal, each being executed on command from the ship; and sixth, a final pointing upon the same adjacent shore station as the first pointing. At the first position of the ship in the example, each of the three stations MUD, TOP, and SIN would make the following observations:

1st part *Tel direct*	*2d part* *Tel reverse*	*3d part* *Tel direct*	*4th part* *Tel reverse*
Initial station	Initial station	Initial station	Initial station
1a	1e	1i	1m
1b	1f	1j	1n
1c	1g	1k	1o
1d	1h	1l	1p
Initial station	Initial station	Initial station	Initial station

At the second position of the ship each of the three stations TOP, SIN, and HAL would make similar observations except that pointings to the ship would be recorded as 2a, 2b, 2c, etc. In the recording for pointings to successive ship positions the lettered pointings are prefixed by the number of the ship position as above.

329. Coordination

To coordinate the observations from each station so that each pointing is made simultaneously by all scheduled stations and to prevent confusion in the recorded values, the following procedure is specified:

a. Upon completion of the craft alinement and upon general report that all scheduled observers are at their stations, the craft commands, "POINT ON INITIAL STATION WITH TELESCOPE DIRECT AND REPORT WHEN READY." It is critically important that observers initial on the proper object. It is all too easy to observe on a wrong object during daylight observations when light conditions may be unfavorable or when backgrounds may cause difficulty in seeing the adjacent shore stations. The use of pocket mirrors, signal lamps, heliotropes, field glasses, and waving of signal cloth are of assistance in locating adjacent stations. One wrong initial pointing may cause the entire shore-ship net to collapse and the cause can be found only after considerable effort. At this time the survey officer in charge on board ship must satisfy himself that the proper initial station is being used. When the initial pointing has been made and recorded, the observer loosens the horizontal clamp and locates the craft in the telescope. Shore stations then report in order of position along the line of advance, with stations to the rear reporting first to avoid confusion. In figure 139, stations would report in the following order: "MUD ready," "TOP ready," "SIN ready," etc.

b. When all scheduled stations have reported, the craft will command, "STAND BY FOR POINTING (1a)" or for whichever pointing is next scheduled. Observers bring the vertical wire to the signal on the ship and track it with the horizontal slow-motion tangent screw. About 5 seconds later the craft will command "READY ONE-TWO-THREE-FOUR-MARK." Observers will keep the vertical wire on the centerline of the signal from the command "READY" and at the command of execution "MARK!" observers will cease tracking the signal, read the horizontal vernier, and again point the telescope on the signal in preparation for the next pointing. *It is important that no attempt be made to better the pointing after "MARK!" is commanded.* If the vertical wire was not on the target at that instant, immediately signal the craft by radio that the pointing was missed. The survey officer in charge on board the craft should likewise make no attempt to better the position of the craft between pointings. A small amount of drift during the course of the 16 pointings is not harmful provided the craft does not drift into a line obstructing the view from a shore station.

330. Missed Pointings

A missed pointing at any one station voids that particular pointing made by other adjacent stations. If any station reports a missed pointing, the craft will command "VOID ALL POINTINGS ON (whichever pointing was missed) AND STAND BY FOR REPEAT POINTING." Then that pointing is started over again. The observer will report a missed pointing, if at the command "MARK!" he has not bisected the craft's target within 1 foot or 0.3 m. (five-sixths of the way off the center of a 3-foot or 1-meter signal). No report is made unless the pointing is missed. If no station reports a missed pointing, the craft will allow about 30 seconds for the shore parties to prepare for the next pointing and then proceed with the commands for pointings (1b), (1c), and (1d), using the same voice procedure as before. During the interval between pointings, it is usually best to track the ship with the plate clamp loose, rather than with the slow motion tangent screw. The screw may be kept centered by training the telescope ahead of the signal and allowing the signal to approach the vertical wire until the command "STAND BY" is given by the craft. It will then usually be necessary to turn the tangent screw in the opposite direction to bring the wire to the target, partially compensating for the turning necessary to follow the target during the tracking procedure.

331. Closures

After completing pointing (1d), the survey officer in charge on the ship will command "CLOSE ON INITIAL STATION AND REPORT WHEN READY." Each shore station then points back to his initial station, reads and records the horizontal circle value; plunges the scope, repoints on the initial station, reads and records the horizontal circle value and then makes, in proper order, either one of the following reports, as appropriate: "STATION ( ) CLOSED OK AND READY"; or: "STATION ( ) CLOSED ( ) SECONDS OUT AND READY." (The selection of the report is determined by whether the horizon closure falls within the specified limits. Maximum allowable error for horizon closure is plus or minus 30 seconds.) As a check on the accuracy of closures, the officer in charge should frequently request information as to how well the horizon closures are coming out. If any station reports a closure beyond the allowable limits, the officer in charge will command that the observing stations reobserve the half-set and proceed with the pointings as before.

332. Reverse

If closures are satisfactory the craft will command "POINT ON INITIAL STATION WITH TELESCOPE REVERSED AND REPORT WHEN READY." Observers will do so, then train the inverted telescope on the initial station and report readiness in order as before. The officer in charge will proceed with commands for pointings and again close on the initial station.

333. Recording

Recorder will compute the mean of the two pointings on the initial station with the telescope direct, subtract this value from each pointing, and record the angle on the right-hand page of the notebook. The reverse or telescope-inverted angles will be extracted in the same manner and the angles compared to determine if any gross error exists. Report of any such apparent errors will be made to the officer in charge, who will determine whether or not the set should be reobserved.

334. Repeat Procedure

The above procedure completes one set of observations. Another set of pointings is then taken in the same manner. The plate on the second set will be set between 90° and 91°; it will not be permissible to take time to set the plates on an even number of degrees, minutes and seconds.

335. Completion

After the completion of two sets, the ship will proceed to the second ship position, from which it will be observed by the two foremost parties of the three used in the first position, and the party next ahead. The rearmost party, upon completion of its work including traverse and picture-point angles (if specified), will proceed to the next unoccupied station ahead. The signal centered over the station will be left standing unless the officer in charge directs otherwise.

Section V. FIELD RECORDS

336. Record Book

All observations taken at a shore station including the *traverse angle* observations as well as the pointings to the ship will be kept in the same book. An example of shore-ship field records is shown in figure 140.

337. Double Coincidence Reading

The observing program for the traverse angle does not include closing the horizon. A double coincidence reading is required on the observations to the adjacent shore stations.

338. Pointing on the Ship

In the pointings on the ship, double coincidence readings are not required. The only mean extracted is the mean of the two pointings on the initial station in a single half set. This mean direction is then subtracted from each pointing to the ship to derive an angle which is used in connection with others to compute a value for the unknown tangent in the shore traverse. The column headed *Forward* is used for recording one-half the difference between the first and last pointing on the initial

HORIZONTAL							DIRECTIONS					
SAMPLE RECORD - SHORE							TRAVERSE OBSERVATIONS					
STATION: △ GUM		OBSERVER: Cpl. C. Mathews; Rec. Pfc. S. Seitz					INSTRUMENT: Wild T-2 # 12027				DATE: 6 Jun 19	
POSITION	OBJECTS OBSERVED	TIME	TEL. D or R	MEAN	°	'	BACK "	FORWARD "	MEAN	MEAN D&R	DIRECTION "	REMARKS
No 1	△ Mud		D	A	00	00	34	34	34.0	33.0	00.0	
			R	B	180	00	32	32	32.0			
	△ Top		D	A	224	39	32	32	32.0	32.0	59.0	224° 38' 59".0
			R	B	44	39	33	31	32.0			
No 2	△ Mud		D	A	45	00	37	37	37.0	37.0	00.0	
			R	B	225	00	37	37	37.0			
	△ Top		D	A	269	39	36	38	37.0	36.0	59.0	224° 38' 59".0
			R	B	89	39	35	35	35.0			
No 3	△ Mud		D	A	90	00	38	37	37.5	36.0	00.0	
			R	B	270	00	36	35	35.5			
	△ Top		D	A	314	39	31	30	30.5	29.2	5.27	224° 38' 52".0
			R	B	134	39	28	28	28.0			
No 4	△ Mud		D	A	135	00	35	36	35.5	36.5	00.0	
			R	B	315	00	37	38	37.5			
	△ Top		D	A	359	39	35	35	35.0	34.8	58.3	224° 38' 58".3
			R	B	179	39	34	35	34.5			
				A								Mean
				B								Mud - Top - 224° 38' 57".25
												Top - Mud - 135° 21' 02".75
				A								
				B								

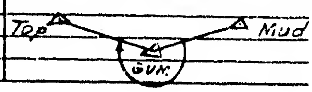


Figure 140. Sample field record of shore-ship observations.

station in the half-set. It is only necessary to carry the determination of each angle to the nearest whole second.

339. Sketch

A sketch of the work performed in each set of notes will be made on the right-hand page of the last group of observations taken for the applicable set. All work on the notes, including sketches, extraction of means, proper headings, and the like, will be completed while the party is in the field and before it leaves the site.

HORIZONTAL DIRECTIONS

SAMPLE RECORD - SHORE-SHIP OBSERVATIONS

STATION: △ GUM OBSERVER: Cpl. C. Matthews REC. Pfc. D. Seitz INSTRUMENT: Wild T-2 # 12027 DATE: 6 Jun 19

POSITION	OBJECTS OBSERVED	TIME	TEL. D or R	MEAN	°	′	BACK ″	FORWARD ″	MEAN	MEAN D & R	DIRECTION ″	REMARKS
	△ Mud		D	A	00	00	25	+3	28			° ′ ″
	8A		D	B	99	40	58					99 - 40 - 30
	8B		D		99	39	49					99 - 39 - 21
	8C		D	A	99	38	55					99 - 38 - 27
	8D		D	B	99	37	55					99 - 37 - 27
	△ Mud		D		00	00	30	-2	28			
				A								
	△ Mud		R	B	180	00	35	-1	34			
	8E		R		279	35	42					99 - 35 - 08
	8F		R	A	279	35	18					99 - 34 - 45
	8G		R	B	279	34	40					99 - 34 - 06
	8H		R		279	34	22					99 - 33 - 48
	△ Mud		R	A	180	00	33	+1	34			
				B								
	△ Mud		D		00	00	29	-1	28			
	8I		D	A	99	32	38					99 - 32 - 10
	8J		D	B	99	32	33					99 - 32 - 05
	8K		D		99	32	26					99 - 31 - 50
	8L		D	A	99	32	32					99 - 32 - 00
	△ Mud		D	B	00	00	33	+1	28			
	△ Mud		R	A	180	00	32	0	32			
	8M		R	B	279	33	44					99 - 32 - 32
	8N		R		279	33	28					99 - 32 - 56
	8O		R	A	279	33	56					99 - 33 - 24
	8P		R	B	279	34	32					99 - 34 - 00
	△ Mud		R		180	00	33	-1	32			
				A								
				B								

DO NOT WRITE IN THIS MARGIN

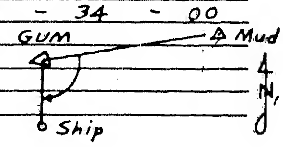


Figure 140—Continued.

Section VI. SPECIFICATIONS

340. Base Lines

Base lines may be of the straight or broken type. In both types, the terminal points must be intervisible. The measurements will be made with such precision that the probable error of the projected length will not exceed 1 part in 70,000; such methods are utilized to insure an actual error no greater than 1 part in 10,000. The projected length of the base line will range from 1 mile to 3 miles (1.6 to 4.8 km). Base lines will be established at intervals of from 30 to 50 miles (50 to 80 km), 40 miles (60 km) being preferable.

341. Azimuth Checks

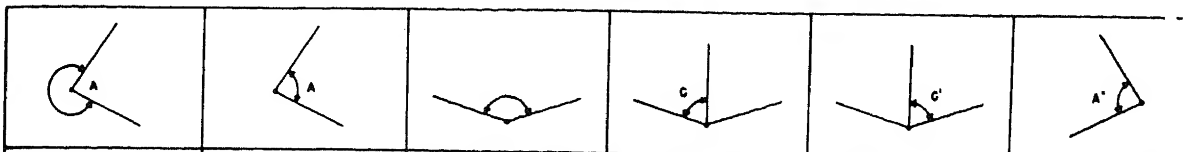
Astronomic azimuth checks for the shore-ship triangulation will be established with the same frequency as for base lines. Azimuth error per station is not to exceed 5 seconds. Astronomic azimuth check lines will be determined with a probable error not to exceed plus or minus 5 seconds.

Section VII. COMPUTATIONS AND ADJUSTMENTS

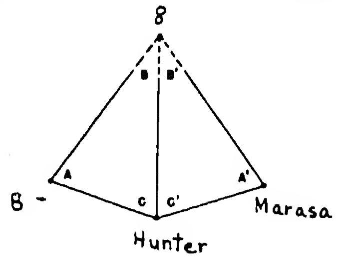
342. Length Computations

Starting from a known length, the lengths of the successive lines of the shore traverse are determined by a series of computations of the shore-ship angles (TM 5-237). For the purposes of the computation it is assumed that there are no instrument errors involved when the method of observing is modified to the extent that single, successive pointings, at a series of signals, indicate the true differences of directions of the signals. (The actual error, in this assumption, normally would not have an effect upon the final result of the work greater,

ABSTRACT OF DIRECTIONS – SHORE-SHIP



RECORDED ANGLE	SHORE-SHIP ANGLE	MEAN SHORE ANGLE	ANGLE AT C	ANGLE AT C'	SHORE-SHIP AN L
° ′ ″	° ′ ″	° ′ ″	° ′ ″	° ′ ″	° ′ ″
326-36-30	33-23-30	135-21-03	99-40-30	124-58-27	35-10-3
326-38-12	33-21-48	" " "	99-39-21	124-59-36	35-08-2
326-39-44	33-20-16	" " "	99-38-27	125-00-30	35-06-5
326-41-20	33-18-40	" " "	99-37-27	125-01-30	35-04-5
326-46-35	33-13-35	" " "	99-35-08	125-03-49	34-59-5
326-48-02	33-11-49	" " "	99-34-45	125-04-12	34-58-2
326-50-31	33-09-29	" " "	99-34-06	125-04-51	34-56-2
326-51-46	33-08-14	" " "	99-33-48	125-05-09	34-55-2
326-59-58	33-00-02	" " "	99-32-10	125-06-47	34-49-2
327-01-45	32-58-15	" " "	99-32-05	125-06-52	34-48-21
327-02-55	32-57-02	" " "	99-31-58	125-06-59	34-47-30
327-04-37	32-55-23	" " "	99-32-00	125-06-57	34-46-29
327-08-32	32-51-28	" " "	99-32-32	125-06-25	34-44-51
327-09-44	32-50-16	" " "	99-32-56	125-06-01	34-44-21
327-10-54	32-49-06	" " "	99-33-24	125-05-33	34-43-59
327-12-24	32-47-36	" " "	99-34-00	125-04-57	34-43-38



SHORE ANGLES AT Hunter	°	′	″
D	224	38	59.0
R	224	38	59.0
D	224	38	60.0
R	224	38	58.0
D	224	38	54.0
R	224	38	51.5
D	224	38	58.5
R	224	38	58.0
SUM	1792	304	458.0
MEAN	224	38	57.25

LOCALITY Guadalcanal Is.

COMPUTED BY H.E.W.

CHECKED BY D.L.T.

REMARKS:

Figure 141. Abstract of directions—shore-ship.

than 1 part in 1,000,000, considering the fact that the horizontal collimation error, by specification has been limited to 10 seconds and that the observed signals lie close to the horizon.) Figure 141 is an example of an Abstract of Directions Form which should be executed before beginning the actual length computations. To maintain a precision of 1 part in 6,000 throughout the shore traverse, the following specifications are listed for the length computations:

a. Angles will be extracted and computed to the nearest second.

b. Logarithms for the the individual computations will be taken to five decimals, using in the computations meaned logarithms to six decimals. The angular arguments for these logarithms will be rounded off to the nearest 10 seconds before entering the tables.

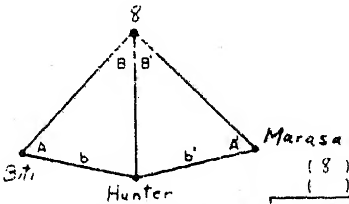
c. The rejection limit for log X (figs. 142 and 143) will be determined by the following rule:

(1) Find the mean of all log X's (including the "wild" one) and find the residual for each. Compute the probable error of a single measurement by the formula

$$E = 0.6745 \quad \frac{\Sigma V^2}{\sqrt{n-1}}$$

(2) Reject any log X whose residual exceeds three times the probable error of a single measurement.

Ship Pos No 8



LENGTH COMPUTATION
SHIP – SHORE

COMPUTED BY H.E.W.
CHECKED BY D.L.T.

FROM MARASA – HUNTER TO HUNTER – BITI
TRAVERSE SECTION # 9 FROM NAGLE TO BOLAVU
PROJECT Guadalcanal LOCALITY Solomons, DATE OF OBS 3 Feb
OBSERVED SHORE TRAVERSE ANGLE 135° 21' 03"

	(8)A ( )I	(8)B ( )J	(8)C ( )K	(8)D ( )L	(8)E ( )M	(8)F ( )N	(8)G ( )O	(?)H ( )P
A ANGLE AT (Marasa)	35-10-33	35-08-28	35-06-55	35-04-55	34-59-58	34-58-28	34-56-26	34-55-28
ANGLE AT (Hunter)	124-58-27	124-59-36	125-00-30	125-01-30	125-03-49	125-04-12	125-04-51	125-05-09
(180°-B)	160-09-00	160-08-04	160-07 25	160-06-25	160-03-47	160-02 40	160-01-17	160-00-37
B ANGLE AT SHIP	19-51-00	19-51-56	19-52-35	19-53-35	19-56-13	19-57-20	19-58-43	19-59-23

A ANGLE AT (Biti)	33-23-30	33-21-48	33-20-16	33-18-40	33-13-35	33-11-48	33-09-29	33-08-14
ANGLE AT (Hunter)	99 40-30	99-39-21	99-38-27	99-37-27	99-35-08	99-34-45	99-34-06	99-33-42
(180°-B')	133-04-00	133-01-09	132-58-43	132-56-07	132-48-43	132-46-33	132-43-35	132-40-02
B ANGLE AT SHIP	46-56-00	46-58-51	47-01-17	47-03-53	47-11-17	47-13-27	47-16-25	47-17-58

LOG SIN A	9.76048	9.76012	9 75985	9.75949	9.75859	9.75832	9.75796	9 75778
LOG SIN B'	9.86366	9 86399	9 86428	9.86458	9.86546	9.86571	9.86604	9 86624
LOG CSC B	0.46908	0.46874	0.46850	0.46815	0 46728	0.46688	0.46641	0.46618
LOG CSC A'	0.25935	0 25967	0.25996	0.26028	0.26124	0.26168	0 26205	0 26231
LOG X (5 PLACES)	0. 35257	0.35252	0.35259	0.35250	0 35257	0 35259	0.35246	0.35251

MEAN LOG X (6 PLACES)	0.352559 (mean of sixteen)
LOG b	3.813372
LOG b'	4.165926

SEE FOLLOWING PAGE FOR METHOD OF DETERMINING REJECTION LIMIT FOR LOG X

UNADJUSTED LENGTH, Hunter TO Biti 14653.00 ± METERS
SUM LENGTH CORRECTION .97
ADJUSTED LENGTH, Hunter TO Biti 14652.03 METERS

Figure 142. Length computation ship-shore.

(3) In the above formula
E = Probable error at a single observation.
V = Residual or algebraic difference between the mean of a set of log X's and the log X for each pointing.
ΣV^2 = Sum of the squares of the residuals.
n = Number of observations.

(4) If more than 3 values for log X are rejected, all will be rejected and that portion of the work will be repeated.

343. Length Adjustment

The error, in closure, of the length of the final line of the shore traverse, as obtained by the shore-ship method, and compared to the length of the measured base, will be distributed throughout the section of the shore traverse involved, as follows:

a. The sum of all lengths determined by the shore-ship method, for that section, is taken and divided into the discrepancy to furnish a correction factor.

b. The correction factor is then used as a constant multiplier for each of the individual lengths; the product of the operation, for each course, furnishing the increment correction to include that course.

c. Starting from the first computed course a continuous sum correction is made for each course; the final course; and the tie-base course which is corrected for the full discrepancy.

d. The above three steps can be expressed as a formula:

Log X	v	v²	
0.35257	+1	1	Ship Pos. No. 8 Probable Error
252	-4	16	
259	+3	9	
250	-6	36	
257	+1	1	
251	-5	25	
246	-10	100	
251	-5	25	
255	-1	1	
256	0	0	
~~274~~	~~+18~~	~~324~~	
260	+4	16	
259	+3	9	
266	+10	100	
256	0	0	
.252	-4	16	
Sum 5 64101		$\Sigma v^2 = 6.79 \times 10^{-8}$	

Mean 0.352563

$$E = 3 \times .6745 \sqrt{\frac{\Sigma v^2}{n-1}}$$

$$E = 2\sqrt{\frac{6.79 \times 10^{8}}{16}}$$

$$E = 2 \times 10^{-4} \sqrt{.4527}$$

$$E = 2 \times .000067$$

$$E = .00013$$

$$\text{Log } x = 0.352551$$

Figure 143. Method of determining rejection factor for log X in the length computation.

$$(1)\quad k = \frac{E}{(L_1 + L_2 + L_3 + \text{------} L_n)}$$

$$(2)\quad C_1 = kL_1,\ C_2 = C_1 + kL_2,\ C_3 = C_2 + kL_3,\ \text{etc.}$$

where: E = Closing error on base, in natural numbers, same unit as the individual lengths.

L_1, L_2, L_3 ----, = Lengths of the shore traverse courses commencing with the first one past the starting base.

L_n = The projected length, carried through the shore-ship work, of the closing base.

k = Correction factor.

C_1, C_2, C_3, ---- = Corrections for the courses.

344. Computation and Adjustment of Shore-Station Azimuths and Positions

After the length discrepancy in the closing length of the shore-station courses has been corrected, the shore-ship survey is reduced to the conditions of a traverse. This traverse, commencing from a station of known position and a line of known azimuth, proceeds along the coast through a series of courses of established lengths; the change of azimuth at each station having been measured and carried forward from the initial azimuth line. The computation and adjustment of azimuths and positions will be executed in the standard method (TM 5-237). All azimuths and positions are geographic, with second-order procedures and specifications utilized throughout. The acceptable error in the final closure in position will be 1 part in 4,000 or less.

345. Conclusions

As a result of experience obtained applying the shore-ship method, the following conclusions are drawn:

a. The method is capable of yielding results

perience ana p

b. It is advantageous to include o tions offshore and to observe closed quadrilaterals whenever terrain conditions permit.

c. On projects where horizontal picture points are required along coast lines only, the method eliminates the necessity of occupying points in the interior sections.

d. Since all survey personnel work closely together as a team, this method eliminates the supply and communications problems resulting from dispersal of parties in conventional survey methods.

e. The use of an LST eliminates, to a great extent, the necessity for the establishment of elaborate base and sub-base camps.

f. Small vessels such as LCM's normally are too unstable to be used as target craft.

g. Since a series of successive short stations, not necessarily occupied, can be observed by this method the picture points themselves can also be included as main-scheme stations, consequently eliminating the necessity, as required in ordinary surveys, of time-consuming work

to tie the picture points into the control.

h. The field methods are such, and the procedure is so designed, as to permit computation to progress along with the fieldwork. This represents a tremendous saving in time over other methods followed in military surveys. The computation, concurrent with the fieldwork, will expose any errors in the fieldwork while the ship and personnel are still in the area.

i. When executing conventional triangulation in tropical islands, personnel are often subjected to unhealthy climatic conditions, primitive shelter, and monotonous diet. In the shore-ship method the use of the ship's quarters and facilities eliminates unsatisfactory field conditions and contributes considerably to the efficiency and morale of the survey personnel.

j. The observational procedure enables the survey officer in charge to supervise closely all personnel involved in the survey and to correct any irregularities in procedure immediately.

k. Every phase of the field survey operations has interrelated, automatic, computation checks.

l. The time required to complete the fieldwork, as compared with normal triangulation procedures, is considerably less than that required under any other method.

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CHAPTER 7
MAGNETIC SURVEYS

346. Introduction

Magnetic surveys are conducted to measure the strength and direction of the earth's magnetic field at specific points on or near the surface of the earth. The earth's magnetic field has an irregular distribution and varies with the passage of time so magnetic measurements must be widely distributed and repeated periodically. Magnetic data are required for the preparation of improved navigation and world magnetic charts, and for other commercial, scientific, and military use.

347. Instruments and Equipment

a. Theodolite-Magnetometer. This instrument is generally referred to as the theodolite-magnetometer since it comprises a theodolite and a magnetometer arranged for mounting on the same base. These two instruments are arranged in this manner since observations to determine magnetic declination consist of two operations—the determination of the true meridian, and the determination of the magnetic meridian. The magnetometer is also used for observations of deflections and oscillations to determine horizontal intensity of the magnetic field.

b. Earth Inductor. This instrument is used to determine magnetic dip and is in principle a small dynamo. Its operation is based on the fact that when a coil of wire forming a closed circuit is rotated in a magnetic field a current of electricity is generated in the coil except when the axis of rotation of the coil is parallel to the lines of force of the field. To determine the magnetic dip with an earth inductor, therefore, it is only necessary to measure, in the plane of the magnetic meridian, the angle of inclination of the axis of rotation of the coil when no current flows as the coil is rotated. The presence or absence of current is indicated by a galvanometer connected to the coil by suitable wiring, through a commutator and brushes which convert the alternating current induced in the coil to a direct current in the galvanometer circuit.

c. Chronometer. A chronometer, adjusted for civil time, is required in magnetic observations, in astronomical observations, and as a timekeeper in determining all elements of the earth's magnetic field.

d. Miscellaneous Equipment. Field equipment for a magnetic observing party should include a nonmagnetic tent, station markers, miscellaneous field tools, and provision for subsisting for a predetermined period.

348. Instrument Standardization

It is important that the results with different field instruments be reduced to a common basis by means [illegible] other or by co[illegible] before and after field [illegible] instruments located at a magnet[illegible] The most desirable method of compa[illegible] instruments is by simultaneous observations at stations not too far apart, with interchange of stations in the middle of the series in order to determine how much, if any, the earth's magnetism differs at the two stations. When simultaneous observations are not possible the records of the instrument's variation may be used to determine the changes in the earth's magnetism between the various times of observation with the two instruments. For direct or indirect comparisons of standard instruments at least 10 sets of observations should be made, but for field instruments at least 6 sets ordinarily will be sufficient. The observation of any one element should be extended over several days, not all made on the same day.

349. Observing Procedures

The site for establishing a station must be free from local magnetic disturbances either natural or artificial. It must be marked for

future recovery and referenced to surrounding landmarks. Where possible, observations are made at a main geodetic survey station, otherwise a true bearing must be obtained by solar or astronomical observations.

a. At a new station, a total of at least four sites should be occupied and at an old station one auxiliary site should be occupied to test the local disturbances. The number of observations increases with the distance the site is removed from an observatory. Within 300 miles (about 500 km) make two sets, 600 miles (1,000 km) make four sets, 900 miles (1,500 km) make six sets, spaced over a period of 10 hours (2 days). Over 900 miles (1,500 km) make 8 sets, distributed so that two are on each of the 4 half-days. The characteristics of the daily change of each element in the region where the observations are to be made should be studied in order to arrange a schedule that avoids measurng an element during hours of most rapid change.

b. The observations should be made inside a nonmagnetic tent or building with the instruments free from the direct rays of the sun. It is preferable to make the observations in the following order.

(1) Declination.
(2) Oscillations.
(3) Deflections.
(4) Deflections (both magnets inverted).
(5) Oscillations (long magnet inverted).
(6) Declination (long magnet inverted).
(7) Dip.

CHAPTER 8
ARCTIC SURVEYING

350. Basic Considerations

This chapter is designed to aid the surveyor in working under the varying terrain and climatic conditions found in the upper latitudes. It describes how he can prepare himself and his equipment for movement from the temperate zone, and discusses problems encountered in the field. The proper use of authorized equipment and field expedients will overcome most problems caused by the cold. For detailed instructions on cold weather indoctrination consult, FM 31-70, FM 31-71, and FM 31-72.

a. Individual Clothing and Equipment. The basis for issue of cold weather clothing and equipment may be found in TA 50-901 and TA 50-902 (clothing and equipment). Mandatory items of personal clothing are listed in AR 700-8400-1. For proper utilization of cold weather clothing and equipment consult FM 31-70.

b. Leadership. Because the effectiveness of equipment is greatly reduced and the demanding requirements on the individual soldier leadership must be of the highest caliber. Individuals should be thoroughly indoctrinated on survival techniques in the Arctic before entering the field.

351. Equipment Preparation

Before the surveying equipment is transported from the zone of interior into the Arctic it must be prepared for the change of climate. It must be properly lubricated, packed, marked, transported, stored, and inspected before being issued to the survey parties.

a. Lubrication. The lubricants normally used in the temperature zone will not stand up under arctic conditions. The proper lubricant for arctic use is—Grease, Artillery and Automotive, Military Specification MIL-G-10924 or equivalent.

b. Packing. Proper packing for shipment is a very important phase of equipment preparation. The basic requirement of any packaging material is protection. In extremely low temperatures discussing surveying in cold weather, the usual instrument packaging which protects against temperature and precipitation is insufficient. Metal tends to shrink in the cold and the firm tight fit can loosen and permit vibrations to develop. One suggested method of protecting against this condition is to encase the instruments in a packaging material which is not susceptible to temperature changes and to high shrinkage in cold weather. Present day cellular or foam plastics offer an excellent material for this protective packaging. Three types in use today are polyethylene, polystyrene, and silicone foams. Polyethylene and polystyrene are the most widely used. Silicone is more highly recommended for
modes of transp
ments are shipped to th
introduce vibration and cause the most
to go out of adjustment. The person responsible for sending these precise instruments to the cold climate should make every effort to see that the proper precautions have been taken to protect the instruments.

c. Marking. The equipment crates should be plainly marked, preferably by stencil. When they reach their destination there should be no doubts as to their contents. "Fragile" labels should be fixed to all crates containing delicate instruments.

d. Transportation. Air transportation is the best method for shipping delicate instruments, since it subjects the instruments to a minimum of vibrations. Crates should be securely fastened to the aircraft to prevent their being jolted on stops, takeoffs, or in air pockets. If the instruments are being transported by ship they should be secured to the deck to prevent the cargo from shifting in transit.

c. Inspection and Storage. When the equipment arrives in the Arctic it should be inspected for damage and then stored in an unheated tent or building before being issued to the units. If at all possible the storage space should be located where it offers the most protection against severe winds. The equipment should get a more thorough inspection by the unit that receives it to make sure it is in proper working condition before being taken into the field.

352. Checkout

Only the equipment needed to complete a job should be checked out. The survey party should not be overloaded with unnecessary items.

353. Repair

Each unit should have at least one qualified instrument repairman (MOS 404). Instruments that cannot be repaired by the unit should be replaced by instruments from storage.

354. Problems in Arctic Surveying

The effects of ice movement, snowfall, prevailing winds, refraction, reflection, glare, and other peculiarities must be studied by survey management when committing survey elements to field operation. Survey accuracy depends largely upon factors that can be controlled in the field by the survey chief. Surveying in the Arctic requires a lot of professional judgment and common sense. Listed below are some facts that could aid the surveyor in making his decisions.

a. Field Operations.

(1) In using angle and direction instruments drive a 3-foot 1-meter metal pipe into the snow for each tripod leg, or if a firmer bottom is desired dig a pit in the snow before sinking the 3-foot (1-meter) pipes, or sink a large timber in the snow and mount the instrument on its top.

(2) Shade the instrument during the whole period, especially if making astronomic observations.

(3) Protect the instrument from the wind or accurate readings will be difficult to make.

(4) Do not make any readings until the instrument has had time to settle (approximately 2 hours).

(5) Do not store instruments in a warm place when not in use. If the instruments are taken from the cold into a heated building and out into the cold again, the moisture in the air will condense on the optics and metal parts. This condensed moisture will then freeze and cause a mist on the optics and a binding on the metal parts. This condition will make observations or readings impossible.

(6) The best way to remedy this condition is to bring the instrument temperature to above freezing to melt the ice and eliminate the moisture. Field expedient methods such as applying heat from light bulbs or candles have been used. This can be dangerous if the heat is applied too close to the frozen optic. If the condensed moisture is on the outside of the optics gently run the orange stick provided in the winterization kit over the optics to absorb the moisture. Do not rub with a cloth.

(7) When transporting instruments from job to job make some arrangement for the instruments to be carried outside the vehicle.

(8) Tripods should also be left outside when not in use.

(9) Astronomic observations will require two tents, one heated tent for the observers to live in and one unheated tent for the instrument while making observations.

b. Safety Precautions.

(1) Do not touch the metal with any part of the bare skin. Make use of the rubber knobs and other accessories provided in the winterization kit.

(2) Make use of any equipment furnished for protection of the eyes against wind and glare.

(3) Always use the buddy system in surveying; do not go out alone.

(4) Always carry a first aid kit.

(5) Practice personal hygiene as covered in FM 31–70.

(6) Learn the proper method of using snowshoes (FM 31–70).

APPENDIX I
REFERENCES

1. Army Regulations

AR 117-5	Military Mapping and Surveying.
AR 320-5	Dictionary of United States Army Terms.
AR 320-50	Authorized Abbreviations and Brevity Codes.

2. Field Manuals

FM 5-1	Engineer Troop Organizations and Operations.
FM 5-30	Engineer Intelligence.
FM 5-146	Engineer Topographic Units.
FM 6-2	Artillery Survey.
FM 21-26	Map Reading.
FM 21-30	Military Symbols.
FM 21-31	Topographic Symbols.
FM 31-70	Basic Cold Weather Manual.
FM 31-71	Northern Operations.
FM 31-72	Mountain Operations.

3. Technical Manuals

TM 5-230	General Drafting.
TM 5-231	Mapping Functions of the Corps of Engineers.
TM 5-232	Elements of Surveying.
TM 5-233	Elements of Surveying.
TM 5-236	Surveying Tables and Graphs.
TM 5-236-year	American Ephemeris and Nautical Almanac.
TM 5-237	Surveying Computer's Manual.
TM 5-240	Map Compilation, Color Separation, and Revision.
TM 5-241-1 through 29	Grids and Grid References.
TM 5-441	Topographic Surveying.
TM 5-6200-1	Alidade, Telescopic (Warren Knight).
TM 5-6300-1	Alidade, Miniature, Telescopic (Dietzgen).
TM 5-6300-2	Level, Dumpy (Dietzgen).
TM 5-6300-3	Transit, Engineer, 1-Minute Reading (Dietzgen).
TM 5-6400-1	Alidade, Telescopic (David White).
TM 5-6400-2	Level, Dumpy (David White).
TM 5-6400-3	Transit, Engineer, 1-Minute (David White).
TM 5-6500-2	Level, Dumpy (Gurley).
TM 5-6500-3	Transit, Engineer, 1-Minute Reading (Gurley).
TM 5-6500-4	Alidade, Miniature, Telescopic (Gurley).
TM 5-6600	Alidade, Miniature, Telescopic (Keuffel and Esser).
TM 5-6600-3	Transit, Engineer, 1-Minute (Keuffel and Esser).
TM 5-6675-200-15	Theodolite, Directional, 1-Minute (T-16).

TM 5-6675-202-15	Microwave Distance-Measuring Unit (Tellurometer).
TM 5-6675-203-15	Altimeter, Surveying (4,500 Meters).
TM 5-6675-204-15	Geodimeter, Mapping and Surveying (30-km Range).
TM 5-6675-205-15	Theodolite, Directional, 0.002 mil (T-2-56-M-MIL).
TM 5-6675-206-15	Geodimeter, Mapping and Surveying (50-km Range).
TM 5-6675-207-15	Gyro-Azimuth Surveying Instrument.
TM 5-6675-210-15	Theodolite, Directional, $\frac{1}{10}$ Second (T4A).
TM 5-6675-211-15P	Alidade, Surveying (Dietzgen).
TM 5-6675-212-15P	Transit, 1-Minute (Dietzgen).
TM 5-6675-213-15	Theodolite, Directional, 1-Second (T-2).
TM 5-6675-214-15P	Alidade, Surveying (White).
TM 5-6675-215-15P	Level, Surveying, Dumpy (White).
TM 5-6675-216-15P	Transit, 1-Minute (White).
TM 5-6675-218-15P	Alidade, Surveying (Keuffel and Esser).
TM 5-6675-219-15P	Alidade, Surveying (Gurley).
TM 5-6675-222-15P	Level, Surveying, Dumpy (Gurley).
TM 5-6675-224-15P	Level, Surveying, Dumpy (Brunson).
TM 5-6675-230-15	Level, Surveying, Precise Tilting (10-X).
TM 5-6675-231-15	Theodolite, Directional, $\frac{2}{10}$ Sec. (T-3).
TM 5-6700-1	Level, Engineer, Dumpy (Brunson).
TM 5-6700-2	Transit, Engineer, 1-Minute (Brunson).
TM 5-9421	Altimeters, Surveying (Wallace and Tiernan).

4. Other Publications

a. Civilian Texts.

Surveying by Breed and Hosmer.
Surveying by Davis and Foote.
Hydraulics Handbook by King.

b. Department of the Interior Publications.

Manual of Instructions for the Survey of the Public Lands of the United States, 1947.

c. U.S. Coast and Geodetic Survey.

SP No. 196 Manual of Tide Observations.
WSP 888 Stream-Gaging Procedure.

APPENDIX II
CONVERSION TABLES

Table XVI. Conversion Factors

Metric to English		English to Metric	
Length			
1 centimeter	= 0.3937 inch	1 inch	= 2.540 centimeters
1 meter	= 3.281 feet	1 foot	= 0.305 meter
1 meter	= 1.094 yards	1 yard	= 0.914 meter
1 kilometer	= 0.621 statute mile	1 statute mile	= 1.61 kilometers
1 kilometer	= 0.5396 nautical mile	1 nautical mile	= 1.853 kilometers
Area			
1 sq. centimeter	= 0.155 sq inch	1 sq. inch	= 6.45 sq centimeters
1 sq. meter	= 10.76 sq feet	1 sq. foot	= 0.0929 sq meter
1 sq. meter	= 1.196 sq yards	1 sq. yard	0.836 sq meter
1 hectare	= 2.47 acres	1 acre	= 0.405 hectare
1 sq. kilometer	= 0.386 sq miles	1 sq. mile	= 2.59 sq kilometers
Volume and Capacity			
1 cu. centimeter	= 0.0610 cu inch	1 cu. inch	= 16.39 cu centimeters
1 cu. meter	= 35.3 cu feet	1 cu. foot	= 0.0283 cu meter
1 cu. meter	= 1.308 cu yards	1 cu. yard	= 0.765 cu meter
1 milliliter	= 0.0338 U.S. liq ounce	1 U.S. liq. ounce	= 29.57 milliliters
1 liter	= 1.057 U.S. liq quarts	1 U.S. liq. quart	= 0.946 liter
1 liter	= 0.2642 U.S. liq gallon	1 U.S. dry quart	= 1.101 liters
1 liter	= 0.908 U.S. dry quart	1 U.S. liq. gallon	= 3.785 liters
1 dekaliter	= 1.135 U.S. pecks	1 U.S. peck	= 0.881 dekaliter
1 hectoliter	= 2.838 U.S. bushels	1 U.S. bushel	= 0.3524 hectoliter
Weight			
1 gram	= 15.43 grains	1 grain	= 0.0648 gram
1 gram	= 0.0353 ounce	1 ounce	= 28.35 grams
1 kilogram	= 2.205 pounds	1 pound	= 0.4536 kilogram
1 metric ton	= 0.984 long ton	1 long ton	= 1.016 metric tons
1 metric ton	= 1.102 short tons	1 short ton	= 0.907 metric ton
Linear Velocity			
1 cm/sec	= 30.48 ft/sec	1 ft/sec	= 3.281×10^{-2} cm/sec
1 m/sec	= 0.3048 ft/sec	1 ft/sec	= 3.281 m/sec
1 m/min	= 0.3048 ft/min	1 ft/min	= 3.281 m/min
1 km/min	= 2.682×10^{-2} miles/hour	1 mi/hr	= 37.28 km/min
1 km/hour	= 1.609 mi/hr	1 mi/hr	= 0.6214 km/hr
1 km/hour	= 3.088×10^{-2} knots	1 knot	= 0.5396 km/hr
		1 knot	= 0.8684 mi/hr
		1 mi/hr	= 1.152 knots

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5–115 (5)
5–129 (10)
5–301 (5)
5–305 (10)
5–327 (5)
5–344 (2)
5–346 (5)
5–348 (15)
5–500 (5)

NG: State AG (3).

USAR: Same as Active Army except allowance is one copy to each unit.

For explanation of abbreviations used, see AR 320–50.

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